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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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ARCH DAMS: MEASUREMENTS AND STUDIES OF BEHAVIOR OF KAMISHIIBA DAM

H. Kimishima,[1] and C. C. Bonin,[2] M. ASCE
(Proc. Paper 1182)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, fifteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin; and "Arch

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Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

The Kamishiiba Arch Dam, 113 meters high from lowest bedrock to crest of roadway, is a unique and precedent-setting structure for Japan. Since it was to be the first arch dam in Japan (in fact in the Far East) its design and construction was the subject of lengthy discussion and controversy among private and government engineers in Japan.

The Kamishiiba Arch Dam was analyzed by the trial load method; modifications to the standard way of applying this method were incorporated in the design. In accordance with standard practice, certain design assumptions were made in order to simplify the laborious computations. Actual conditions while similar, are different from those assumed. This results in actual behavior somewhat different from that computed.

For this reason, approximately 480 different types of Carlson meters were embedded in the concrete and rock foundation, to be utilized for investigation of the actual behavior of this thin arch structure. This paper describes the results of preliminary investigations, utilizing data obtained from these meters; the paper also discusses the determination of elastic properties, temperatures, deformations and stresses in the concrete and foundation rock.

Before the completion of joint grouting operations and before all concrete was placed, the Kamishiiba Dam was subjected to two unscheduled temporary water loadings. These resulted from typhoons that increased the flow of the river far beyond the diversion tunnel capacity. In each case, the reservoir rose very quickly behind the dam to nearly two-thirds of its design head. These events presented a very unusual opportunity to measure stresses and deflections in a structure of this type. This opportunity was not lost and readings were taken of all the embedded instruments at the time of these loadings. These readings made possible a number of the conclusions drawn in this paper.

Arrangement and Method of Measurement

For a structure as large as the Kamishiiba Dam the problem involved is how to arrange a necessarily limited number of meters in the proper locations. The basic principles adopted at Kamishiiba were as follows:

1. In order to obtain an overall picture of the behavior of the dam - even if there be a slight sacrifice in accuracy - meters must be distributed over the entire structure, since assumptions of homogeneity and geometrical symmetry were known not to be entirely true as far as the actual behavior of the dam was concerned.

2. The location of the meters must be closely correlated to the respective elements (arch and cantilever) utilized in the analysis by trial load method.

3. Strain meter readings must be checked to some degree, either by duplication or by stress meters.

4. In locations where high stresses are expected, or in locations of special interest, measurements must be concentrated to some extent in order to be certain of the accuracy.

Location of the meters as utilized at Kamishiiba is shown in Figures 1 and 2.

The Kamishiiba Dam, being a relatively thin structure, does not include an inspection gallery. For this reason it was necessary to bring every cable from the embedded meters to terminal boxes on the downstream face of the dam. Readings are made with portable test sets at specified intervals, from catwalks provided on the downstream face.

Preliminary Investigations

The standard tests on the various properties of cement, rock and concrete were made; these investigations are summarized in Table 1 and Figure 3. In the general area of the dam site a number of classifications of bedrock were found. These were all sampled and subjected to physical and chemical tests. They were designated from Class A to Class D, ranging from the most excellent to extremely weathered graywacke. Some of the results of these tests are indicated in Table 2. This table indicates that the rock itself is extremely hard; it should also be noted that there are many minute cracks and fissures in this hard foundation rock.

Typical concrete mixes, their strengths and instantaneous moduli of elasticity are shown in Tables 3 and 4. These test investigations were made prior to the actual construction.

Measurements of creep, instantaneous modulus of elasticity and coefficient of thermal expansion were made on copper-sealed 8-in. by 16-in. cylinders, utilizing Carlson strain meters and car springs. The results of these measurements are shown in Figures 4, 5 and 6. Poisson's ratios were measured on approximately 300 cylinders, 6 in. by 12 in., by attaching dial gage extensometers. Values obtained by these tests indicate little or no difference resulting from age or cement content; the average value of Poisson's ratio was 0.16. The thermal properties of the concrete to be utilized at the dam were measured by conventional methods and the results obtained are as follows:

1. Thermal conductivity equals 0.00480 cal/cm, sec^{o}C

2. Specific heat equals 0.23 cal/gr, ^{o}C

3. Thermal diffusion equals 0.0088 cm^2/sec

Autogenous growth of the concrete was measured by embedding non-stress meters in the dam; growths were measured ranging from 30 x 10^{-6} to 70 x 10^{-6} in. per in. for the first 6 months after the concrete was placed.

Deformations of the Foundation Rock

Since the arch dam is an indeterminate structure, deformations of the foundation rock are one of the designers' greatest uncertainties. As indicated

FIGURE 1

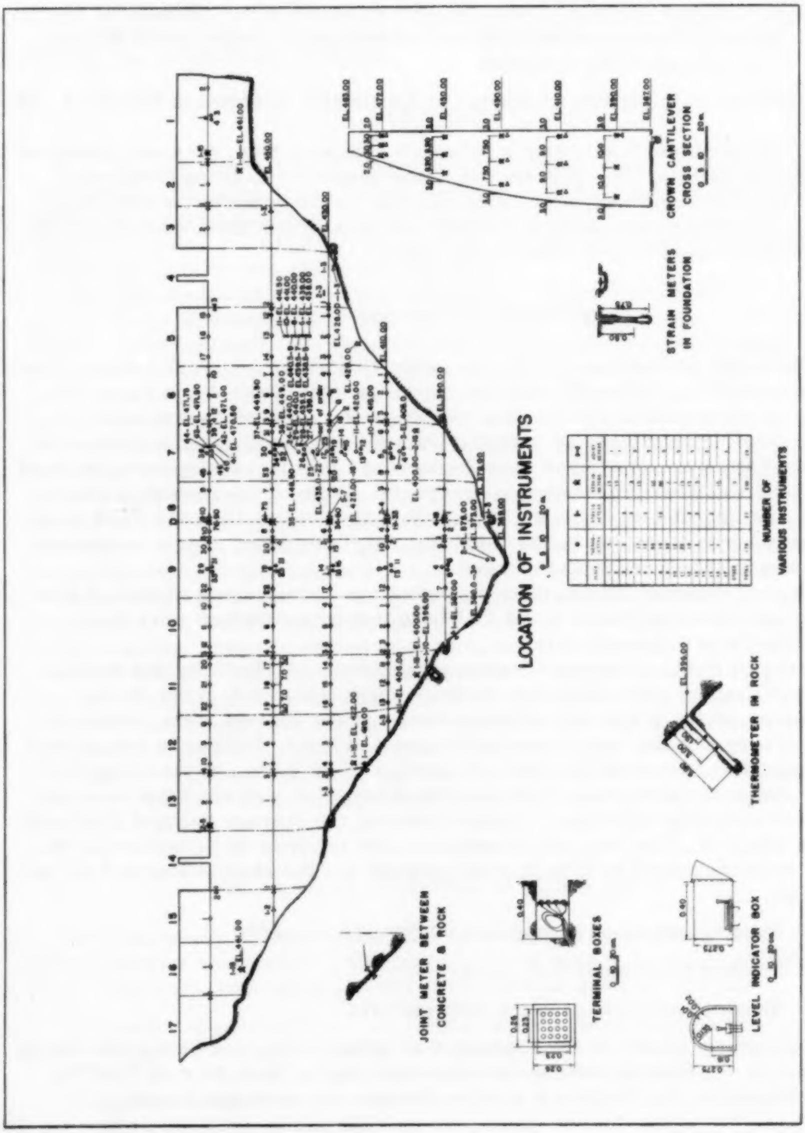

LOCATION OF INSTRUMENTS
CROWN CANTILEVER CROSS SECTION
STRAIN METERS IN FOUNDATION
NUMBER OF VARIOUS INSTRUMENTS
THERMOMETER IN ROCK
JOINT METER BETWEEN CONCRETE & ROCK
TERMINAL BOXES
LEVEL INDICATOR BOX

FIGURE 2

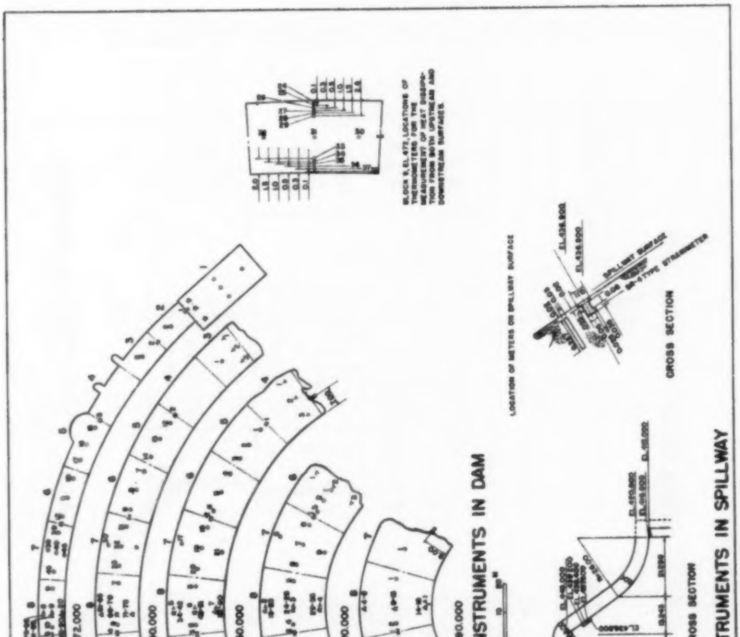

EL.472.000
EL.450.000
EL.430.000
EL.410.000
EL.390.000
LOCATION OF INSTRUMENTS IN DAM
CROSS SECTION
PLAN
CROSS SECTION
LOCATION OF INSTRUMENTS IN SPILLWAY

TABLE 1

CHARACTERISTICS OF CEMENT USED IN KAMISHIIBA DAM

Chemical Composition

	Specification %	Random Samples (%)
Loss on ignition	< 2.0	0.82
Insoluble residue	< 0.75	0.42
Silicon dioxide (SiO_2)	-	23.92
Aluminum oxide (Al_2O3)	-	4.22
Ferric oxide (Fe_2O_3)	-	4.10
Calcium oxide (CaO)	-	63.50
Magnesium oxide (MgO)	< 3.0	1.46
Sulfuric anhydride (SO_3)	< 2.0	1.26
Tri-calcium silicate ($3.CaO.SiO_2$)	<50.0	38.8
Di-calcium silicate ($2CaO.SiO_2$)	-	39.4
Tri-calcium aluminate ($3CaO.Al_2O_3$)	< 7.5	4.3
Heat of hydration- 7 days	<70 cal/g	57.8
28 days	<80 cal/g	71.4

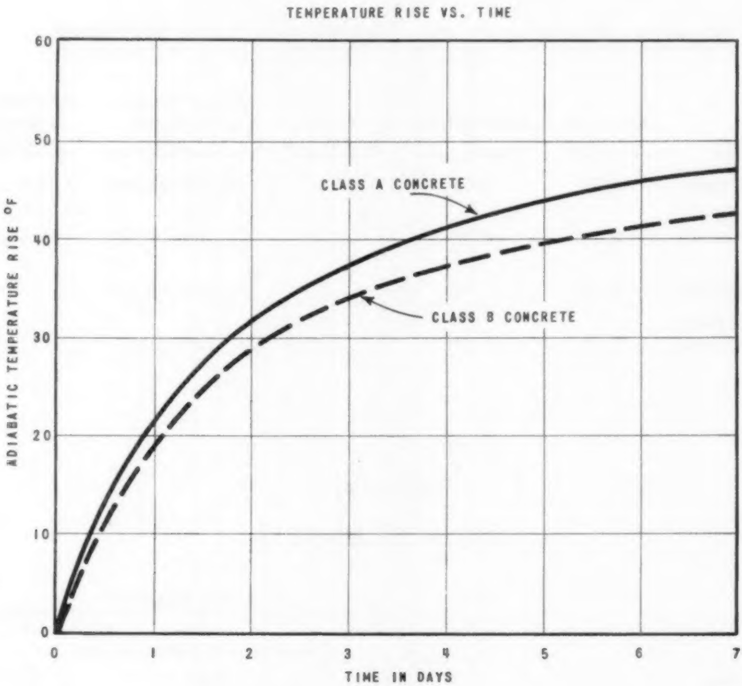
Physical Properties

		Specification	Random Samples
Specific gravity		J.I.S.	3.20
Fineness (cm^2/g)		> 1700	1830
Soundness (%)		< 0.5	0.04
Initial set (hr)		J.I.S.	2^h-14^m
Final set (hr)		J.I.S.	3^h-22^m
Compressive Strength	3 days (psi)	> 570	1230
	7 days (psi)	> 1290	2100
	28 days (psi)	> 3140	4200
	91 days (psi)	> 4600	6800
Bending Strength	3 days (psi)	J.I.S.	340
	7 days (psi)	J.I.S.	530
	28 days (psi)	J.I.S.	850
	91 days (psi)	J.I.S.	1100

J.I.S.: Japan Industrial Standards

FIGURE 3

ADIABATIC TEMPERATURE RISE OF THE SPECIFIED CONCRETE



PROPORTION OF THE CONCRETE

MIX / CLASSES	CEMENT CONTENT SACKS PER CUBIC YARD	W/C (%) BY WEIGHT	SAND POUNDS PER CUBIC YARD	COARSE AGGREGATE POUNDS PER CUBIC YARD
A	4-1/2	0.48	1070	2390
B	4	0.53	1090	2420

TABLE 2

ELASTIC PROPERTIES OF GRAYWACKE (2 TO 4 INCH CUBES)

Classes	Specific Gravity	Absorption %	Shore Hardness	Compressive Strength (psi)	Modulus of Elasticity (psi)
A (excellent)	2.72	0.04	82.5	38,000-27,000	11.4×10^6
B (partially weathered)	2.65	0.98	69.96	30,000-19,000	8.5×10^6
C (semi-weathered)	2.60	1.44	-	24,000-10,000	-
D (completely weathered)	2.53	3.69	-	17,000-2,400	-

TABLE 3

CONCRETE MIXES

	Concrete Classes		
	A	B	C
	(pounds per cubic yard)		
Cement	395	353	320
Water	204	202	210
Sand	870	875	970
1/4" - 3/4"	518	525	500
3/4" - 1-1/2"	518	525	500
1-1/2" - 3"	700	785	830
3" - 6"	700	785	680
W/C Ratio by weight (%)	51.5	57.1	65.8

Maximum size of aggregate: 6 inches
All aggregates were crushed sandstone.

FIGURE 4

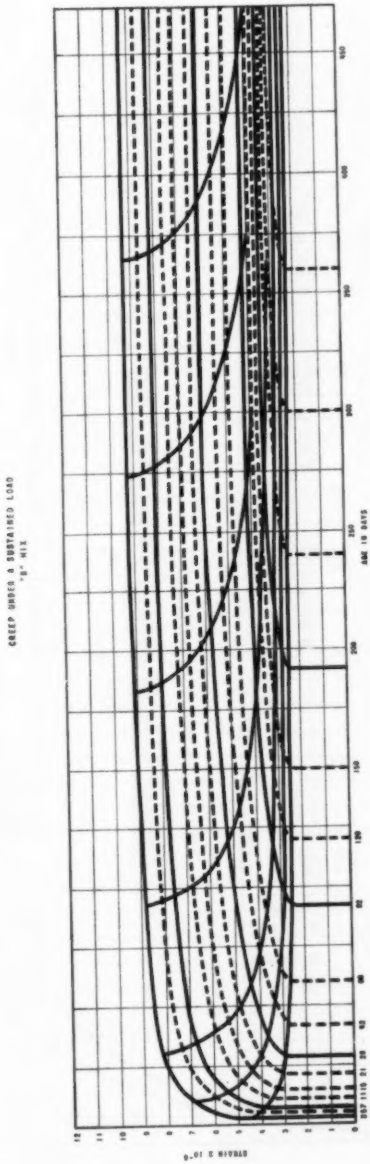


TABLE 4

STRENGTH AND INSTANTANEOUS MODULUS OF ELASTICITY OF CONCRETE USED IN KAMISHIIBA DAM

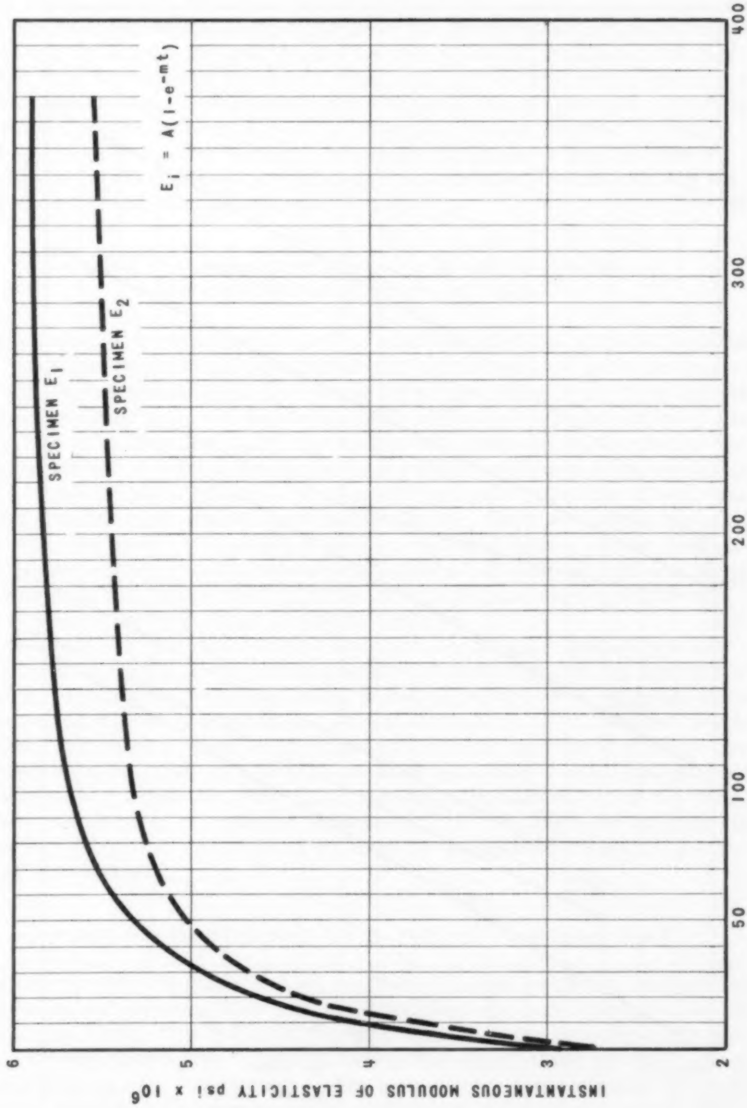
	Concrete Classes A	Concrete Classes B	Concrete Classes C
	(pounds per square inch)		
7 days	2150	1800	1390
28 days	3950	3700	2800
91 days	5330	5250	4100
E_i @ 7 days	4.97×10^6	4.82×10^6	4.25×10^6
E_i @ 28 days	5.38×10^6	5.25×10^6	5.1×10^6

TABLE 5

EFFECTIVENESS OF ARTIFICIAL COOLING

	Maximum Temperature Rise	Age at the Maximum Temperature	Ratio of the Max. Temperature	Rate of Temperature Drop Max. = 100% Age In Days: 20	30	50	60	70	Thickness
Pipe Cooling	$44^\circ F$	3 days	0.8	60%	40%	25%	12%	0%	54 feet
Natural Cooling	$55^\circ F$	30 days	1.0	97%	100%	93%	88%	78%	35 feet

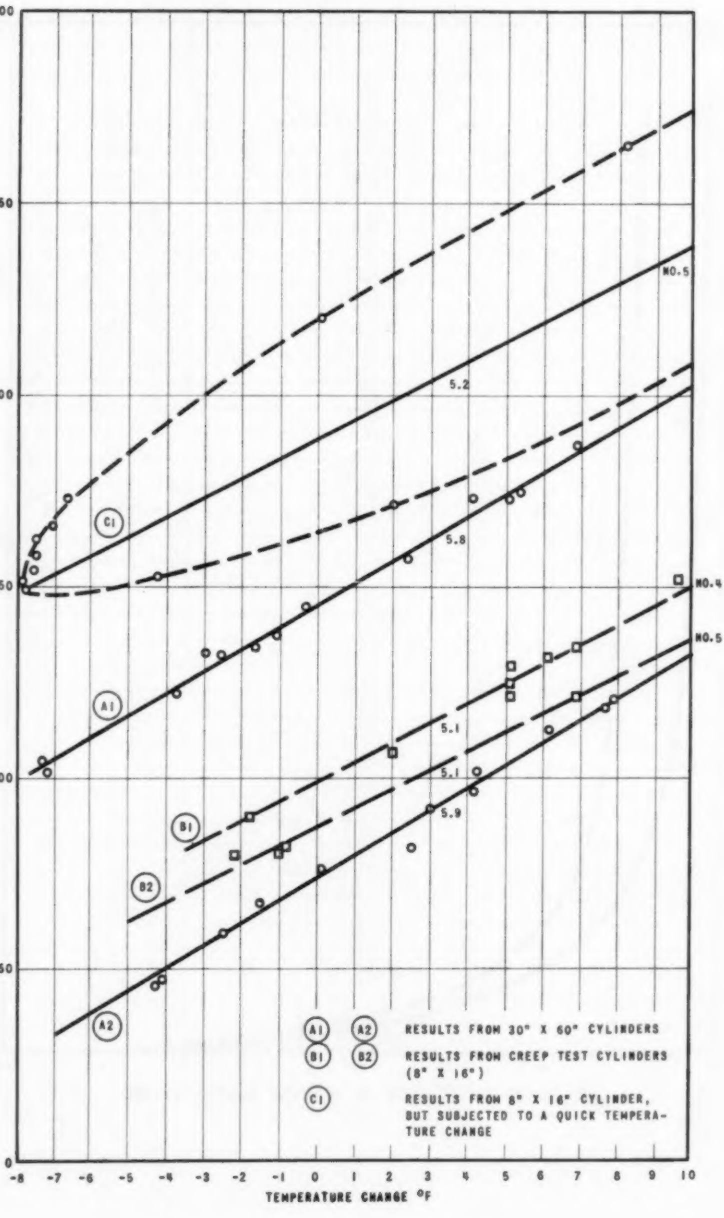
FIGURE 5.



INSTANTANEOUS MODULUS OF ELASTICITY VS. DAYS
(BY 12"x24" CYLINDERS)

FIGURE 6

COEFFICIENT OF THERMAL EXPANSION



in the paper outlining the design of Kamishiiba Dam, the effects of various assumptions concerning these deformations were studied. At the same time it was considered necessary to determine, to the greatest possible degree of accuracy, the actual deformation characteristics of the rock at the site. Measurements were made both by mechanical jack and by the use of embedded strain meters; deformations were determined and moduli of elasticity of the foundation as a whole were computed by use of Boussinesque's formula. Results varied widely, depending upon the direction and location of the test load and upon the slope of the curve drawn through the test points showing deflection against load. The moduli ranged from 0.18 x 10^6 psi to 8.5 x 10^6 psi. Figure 7 shows the deformation of the foundation at one location as determined by the jack-loading method. This wide variation in the modulus of elasticity of the foundation rock is one of the important factors causing the actual stress to vary from those calculated.

Actual Opening at Contraction Joints and at Contact Surface Between Concrete and Rock

The extensive system of joint meters made possible the continual readings of the joint openings and the openings at the contact surface between concrete and rock; the variations in these openings are illustrated in Figures 8, 9, 10 and 11. The behavior of the joints are self-evident from examination of these figures; the variations are fairly consistent with concrete temperatures except as modified by other factors, such as grouting operations and unexpected water load placed upon the dam. In general, and with respect to the particular joint openings pictured on the attached figures, the normal joint grouting pressure acting upon the joints opened them nearly 0.012 in., while the temporary water loading closed them approximately 0.004 in. In both instances there was an elastic movement, with the structure soon recovering to its initial state leaving only slight permanent effects.

However, at the joint between the concrete and rock it appears that the contact grouting had a considerably greater effect on the opening than was the case at a contraction joint; in addition, it was noted that at these joints some permanent movement was retained. On the other hand, the movement due to the temporary water load appeared to be of a permanent nature at these contact joints even in those areas where the load was applied after contact grouting had been accomplished. Generally speaking, the deflections of the arch dam were sensitive to all external pressures; this results in the obvious conclusion that extremely careful control must be maintained during all joint grouting operations.

Temperature Observations in Concrete and Foundation Rock

Resistance thermometers as well as other Carlson meters were installed to determine temperature distributions throughout the dam structure and in the foundation rock. The question of temperatures in the structure was one of great concern during the course of the construction, since the dam was built utilizing 2 meter lifts having a plan area for the largest block of nearly 680 square meters. The continuous measurement and analysis of the temperatures were necessary to determine the effectiveness of the pipe cooling, to make decisions concerning the rate of concrete placement, and to decide

FIGURE 7

DEFORMATION OF FOUNDATION ROCK

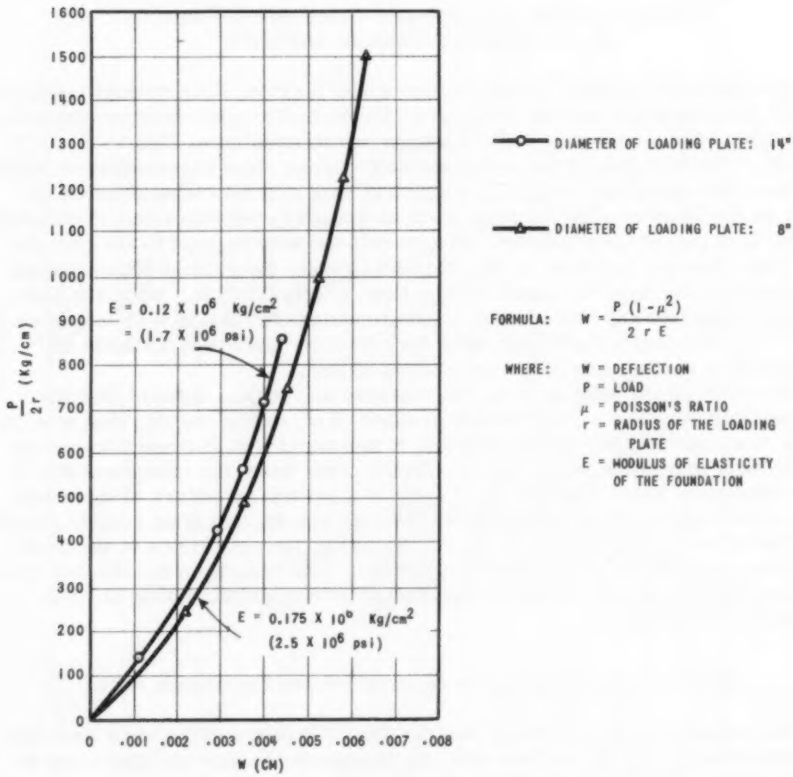


FIGURE 8

JOINT OPENING
ELEV. 430^{m} JOINT 6-7

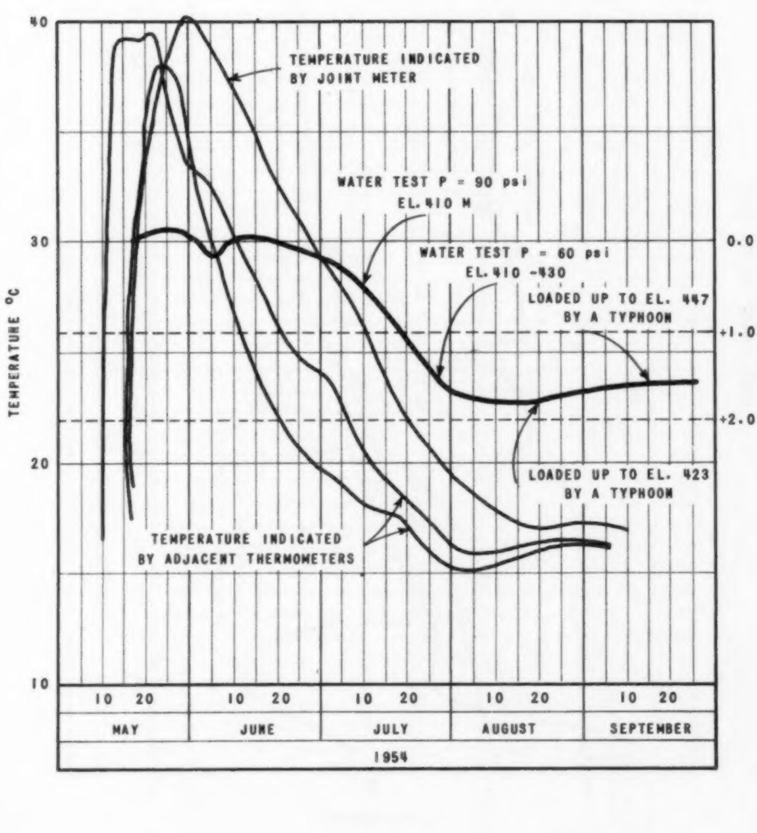


FIGURE 9

JOINT METER READING
ELEV. 410m JOINT 8-9

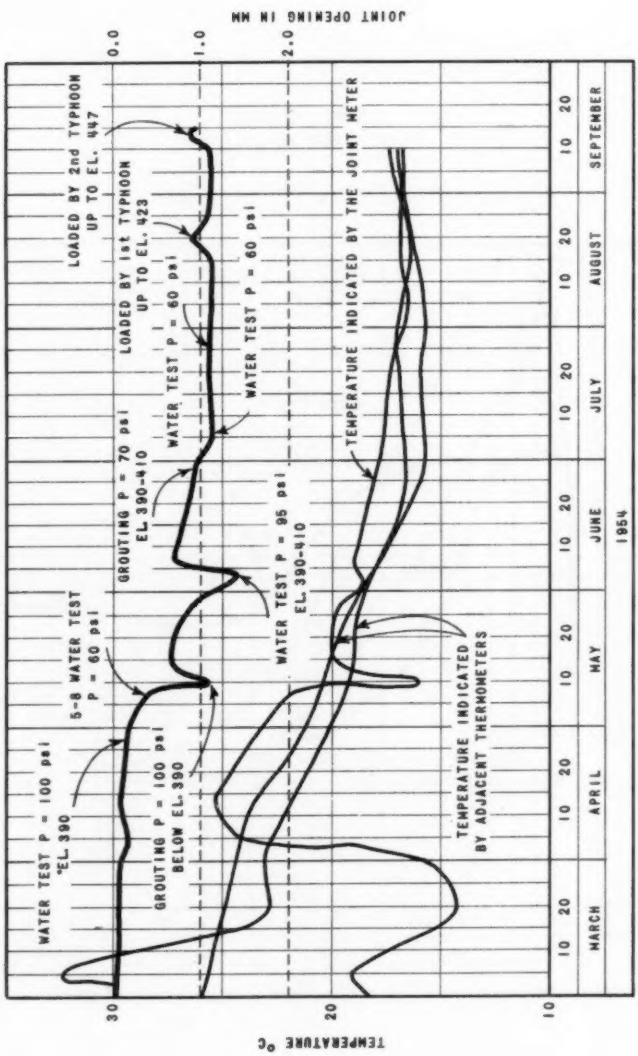


FIGURE 10

JOINT OPENING BETWEEN CONCRETE AND SLOPED FOUNDATION ROCK
BLOCK 10 ELEV. 387.00m

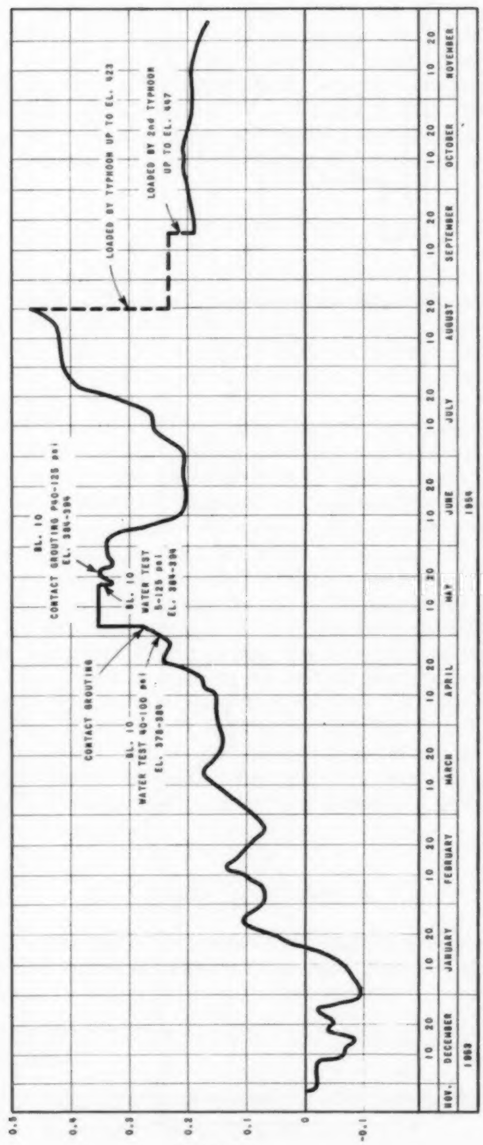
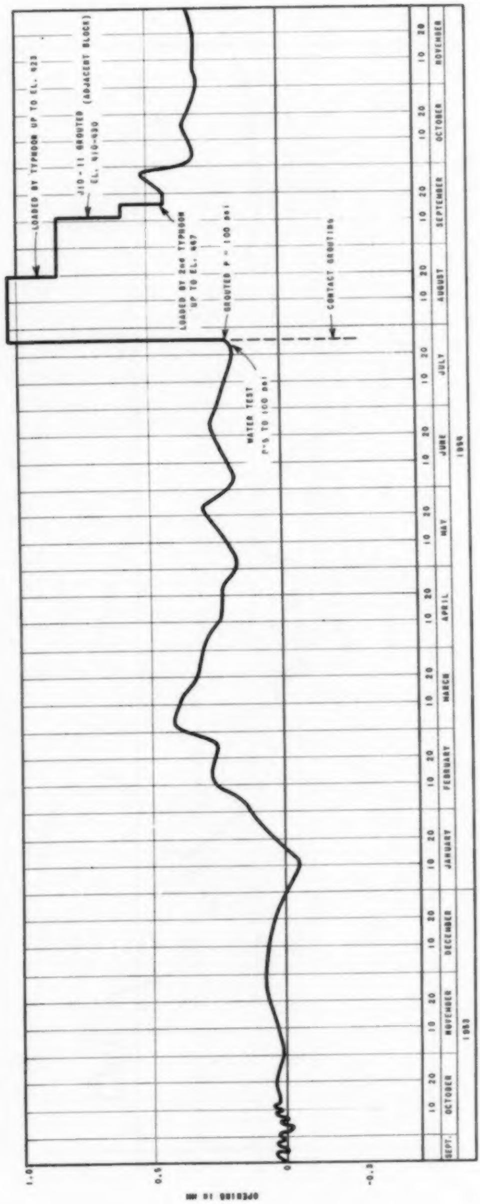


FIGURE 11



JOINT OPENING BETWEEN CONCRETE AND VERTICAL FOUNDATION ROCK
BLOCK 12 ELEV. 410.00

upon the joint grouting schedule. Figures 12 and 13 show temperature distributions in the concrete and the foundation rock at various ages. The effect of a facing concrete with higher cement content than the interior concrete can be seen from the steep gradient shown on Figure 12. Figure 14 shows the daily and hourly variations of temperature near both faces of the dam; the effect of solar radiation on the concrete at varying distances from the face can be seen in this figure. Figure 15 and Table 5 illustrate the marked effect of the pipe cooling adopted in the different sections of the dam; temperature gradient was greatly reduced by the use of artificial cooling.

Measurement of Deflection

In order to measure the deflection of the dam both during construction and after filling of the reservoir, accurate level indicators were installed on the downstream face of the dam at various locations. The accuracy of these indicators is such that a minimum reading of 0.00005 radians can be made. With the indicators installed at every 20 meters in height, a deflection of one millimeter can be read.

Figure 16 shows deflections at the crown section. Unexpected typhoon loadings (which occurred during the joint grouting operations of the lifts between Elevation 410 and 430 meters) caused a large deflection downstream at Elevation 430; however, this deflection was almost entirely recovered by the subsequent joint grouting operations which actually left a counter-deflection below Elevation 410 meters. During the period November 1954 to April 1955 the effects of joint grouting and temperature shrinkage tended to compensate each other - resulting in a slight downstream deflection at the end of this period, except for a slight upstream deflection at Elevation 410.

After the normal reservoir filling began, the deflection of the crown cantilever increased as the reservoir rose to its normal maximum level in October 1955; thereafter, the deflections continued to increase even though the reservoir level was lowering. It appears from these latter readings that the temperature effect on the deflection counterbalanced the water load effect. Figure 16 also shows that actual deflections are less than were computed. Although not illustrated, there was little or no variation in joint openings due to variations in water load.

Arch Stress in the Dam

By utilizing both stress and strain meter readings it was possible to determine the actual arch stresses at various locations. Figures 17, 18 and 19 illustrate respectively:

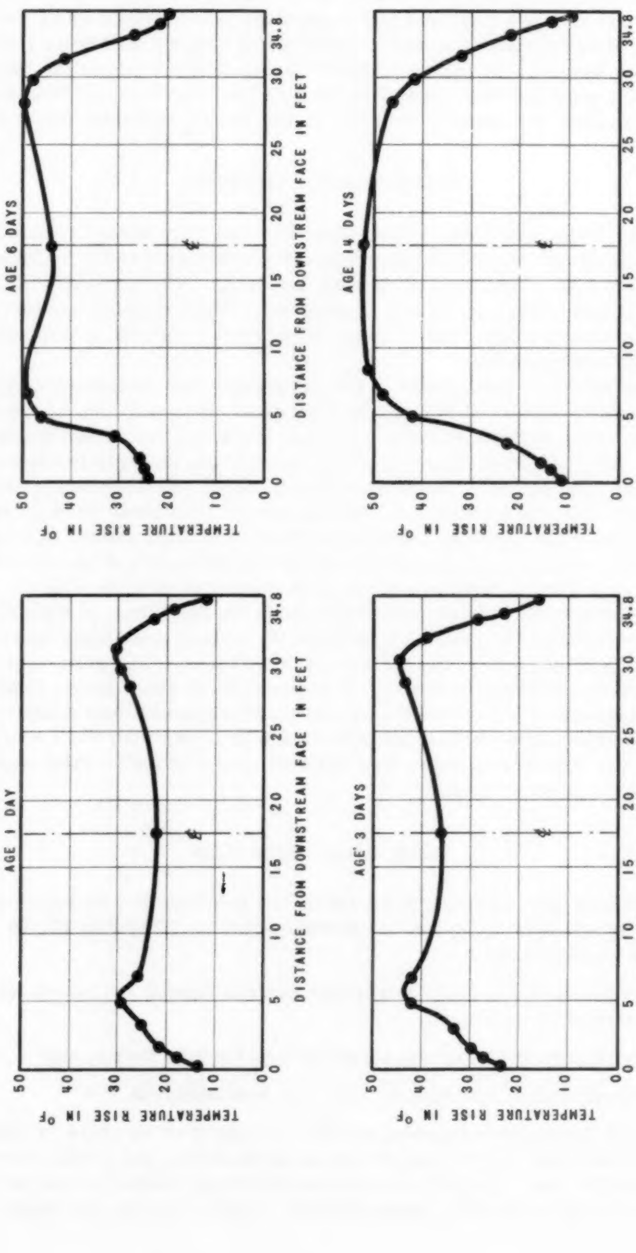
1. elevations of the reservoir water surface during the course of the loading of the dam,

2. arch stresses at various locations and various times, and

3. computed arch stresses by the trial load analysis.

Analyses of these three figures, and the calculations involved in making them, show that in a dam of the proportions of Kamishiiba, the initial stress existing before the water loading occurs is a relatively large portion of the total stress in the dam after the water loading. These figures also show that the

FIGURE 12
1 OF 2



TEMPERATURE DISTRIBUTION ACROSS A SECTION AT VARIOUS AGES
BLOCK 9, THICKNESS 34.8 FEET, NATURAL COOLING

FIGURE 12
2 OF 2

TEMPERATURE DISTRIBUTION ACROSS A SECTION AT VARIOUS AGES
BLOCK 9, THICKNESS 34.8 FEET, NATURAL COOLING
(CONTINUED)

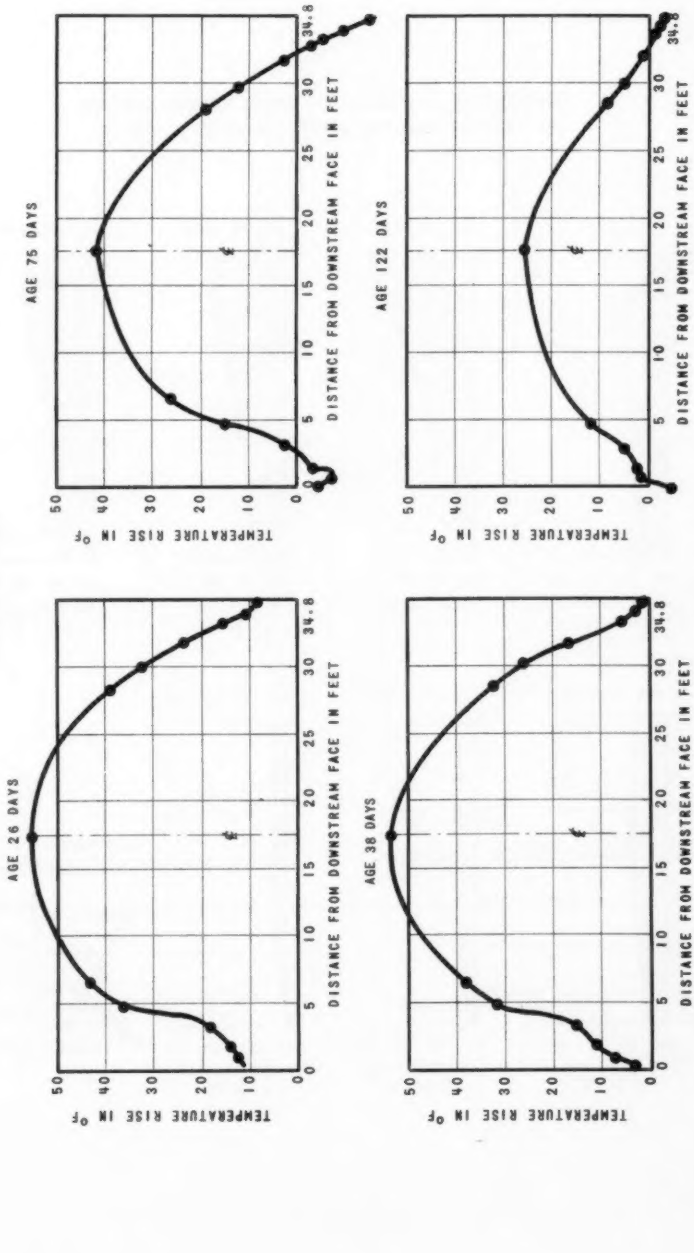


FIGURE 13

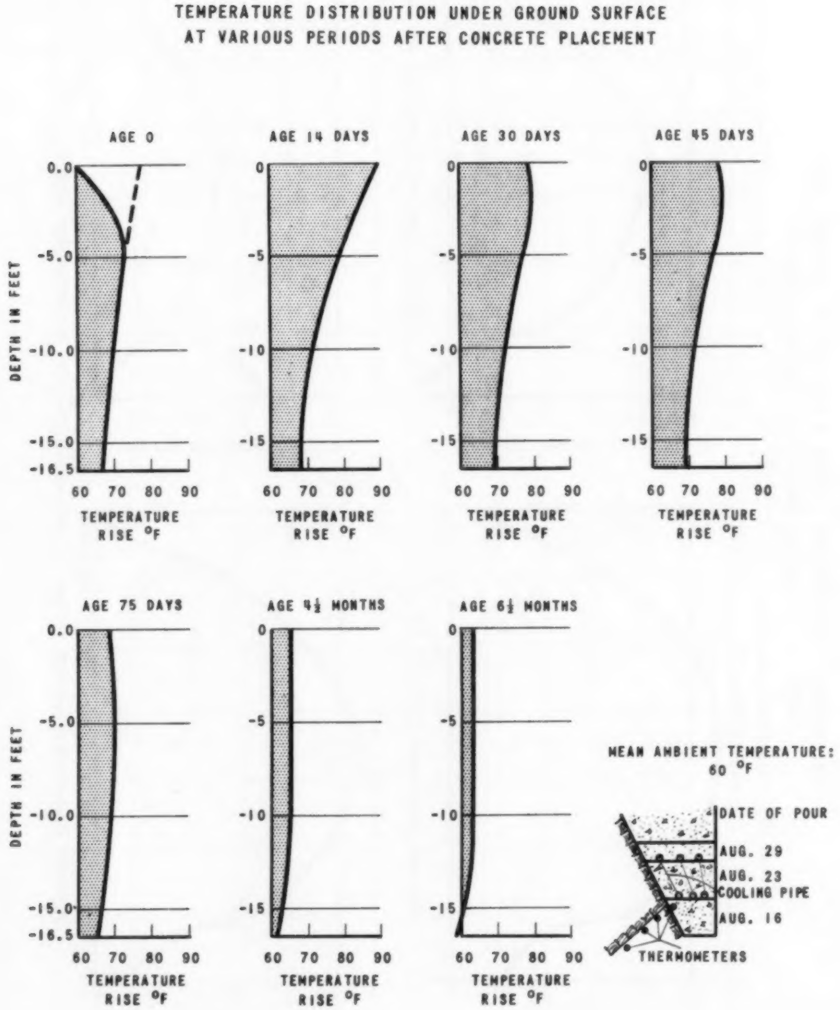

TEMPERATURE DISTRIBUTION UNDER GROUND SURFACE
AT VARIOUS PERIODS AFTER CONCRETE PLACEMENT
AGE 0
AGE 14 DAYS
AGE 30 DAYS
AGE 45 DAYS
AGE 75 DAYS
AGE 4½ MONTHS
AGE 6½ MONTHS
DEPTH IN FEET
0.0
-5.0
-10.0
-15.0
-16.5
60 70 80 90
TEMPERATURE RISE °F
MEAN AMBIENT TEMPERATURE: 60 °F
DATE OF POUR
AUG. 29
AUG. 23
COOLING PIPE
AUG. 16
THERMOMETERS

FIGURE 14

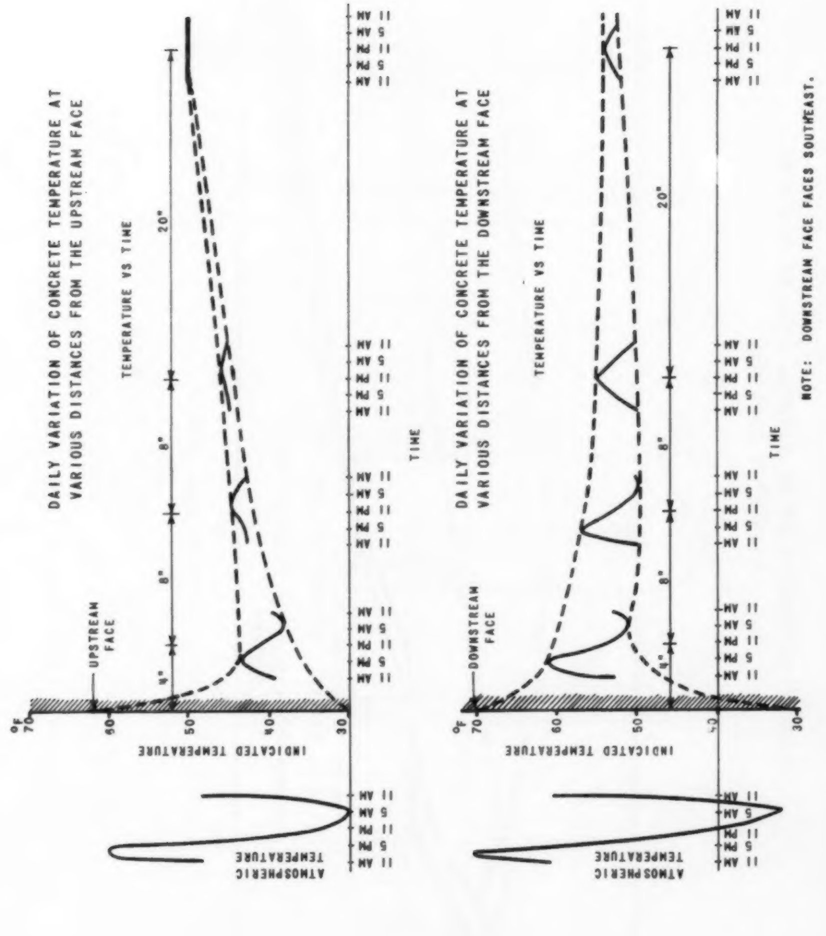

DAILY VARIATION OF CONCRETE TEMPERATURE AT
VARIOUS DISTANCES FROM THE UPSTREAM FACE
TEMPERATURE VS TIME
UPSTREAM FACE
ATMOSPHERIC TEMPERATURE
INDICATED TEMPERATURE
°F
70
60
50
40
30
4"
8"
8"
20"
11 AM
5 PM
11 PM
5 AM
TIME
DAILY VARIATION OF CONCRETE TEMPERATURE AT
VARIOUS DISTANCES FROM THE DOWNSTREAM FACE
TEMPERATURE VS TIME
DOWNSTREAM FACE
NOTE: DOWNSTREAM FACE FACES SOUTHEAST.

FIGURE 15

TEMPERATURE VARIATION BY NATURAL COOLING COMPARED
WITH PIPE COOLING

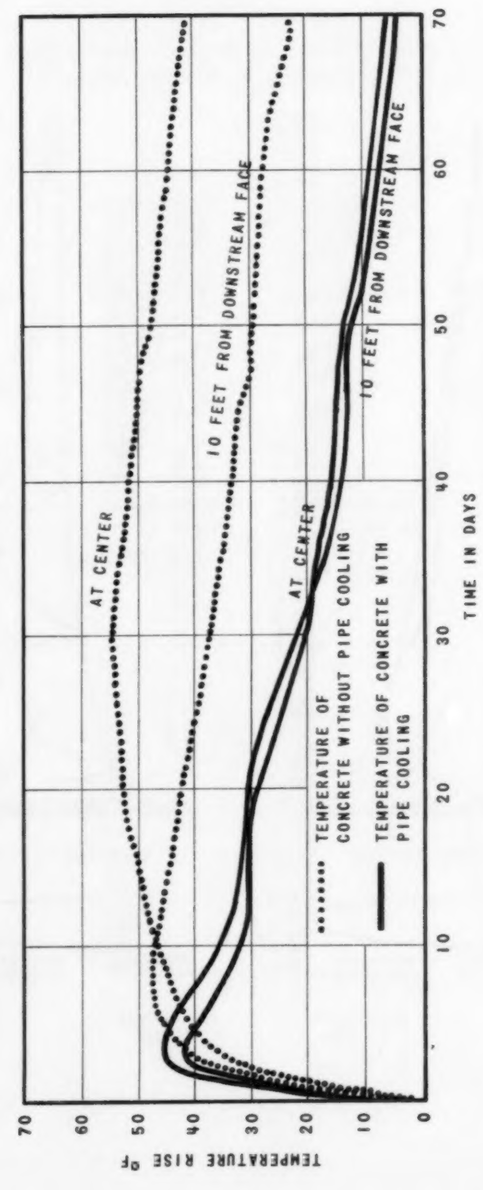


FIGURE 16
1 OF 2

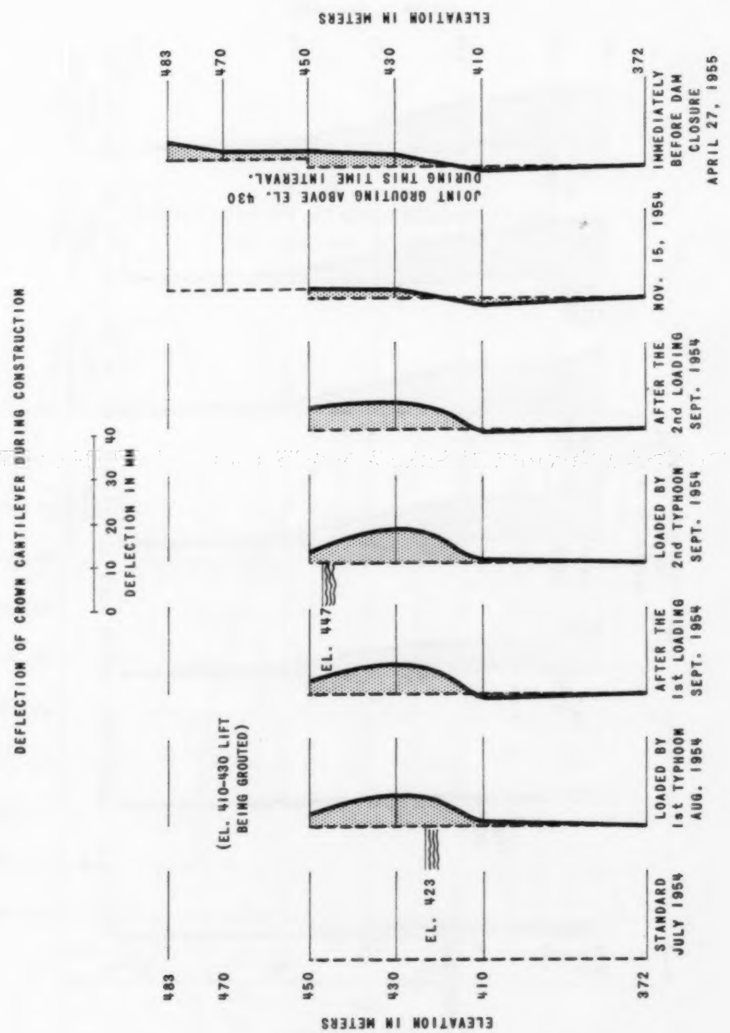


FIGURE 16
2 OF 2

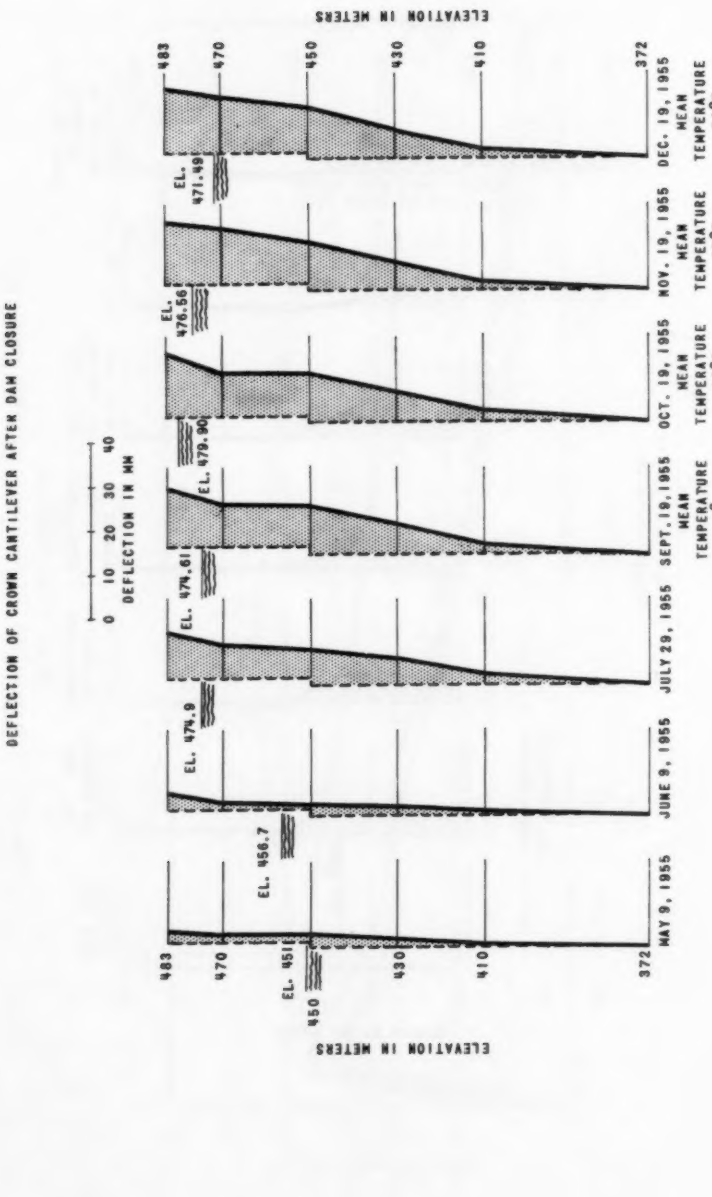


FIGURE 17
1 OF 2

ELEVATIONS OF WATER SURFACE AFTER
THE LOADING STARTED

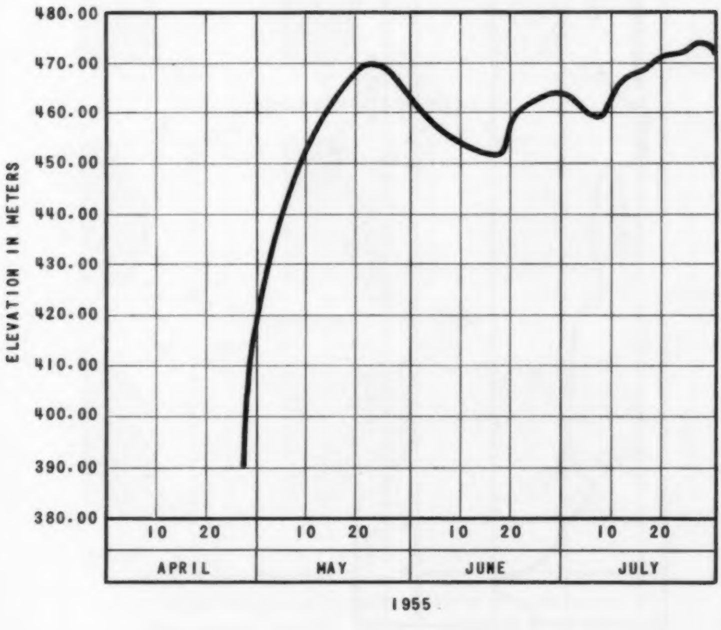


FIGURE 17
2 OF 2

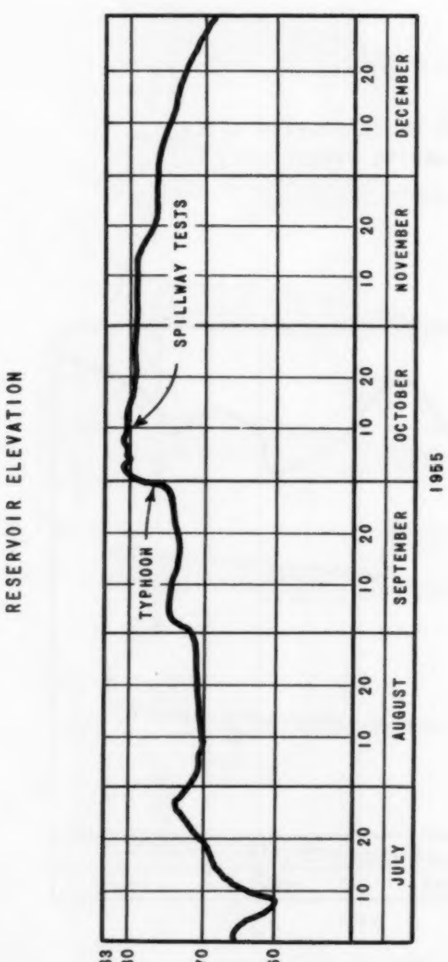
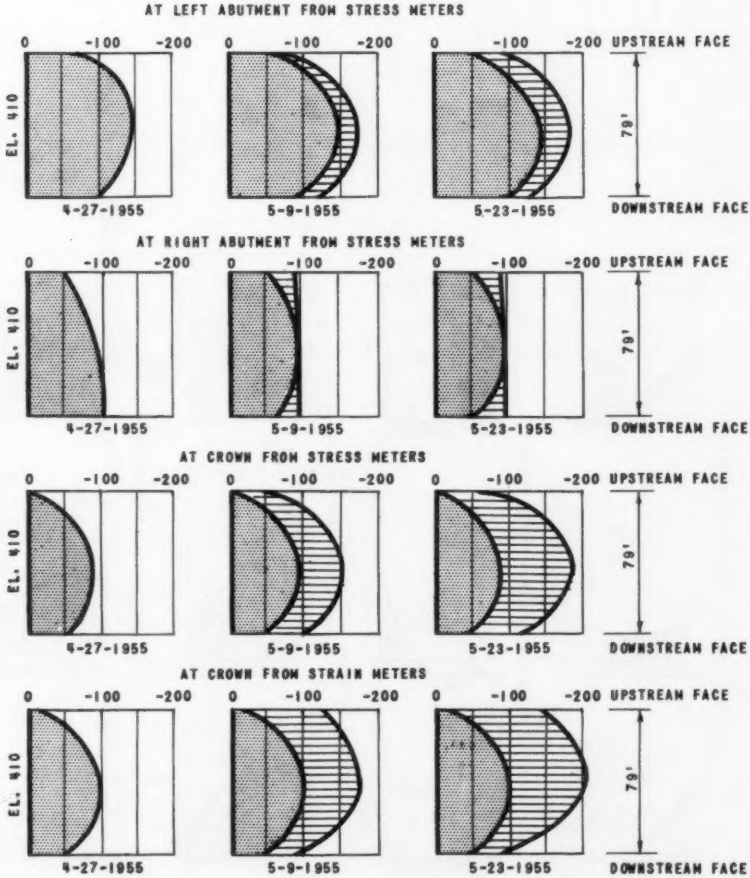


FIGURE 18
1 OF 6

ARCH STRESSES AT VARIOUS POINTS



NOTES:

FIGURES SHOW STRESSES IN psi

+ INDICATES TENSILE STRESSES, - INDICATES COMPRESSIVE STRESSES

BLACK SHADOWS INDICATE INITIAL STRESSES EXISTED BEFORE CLOSURE OF DAM

HATCHED LINES INDICATE STRESSES DUE TO EXTERNAL LOADS AFTER CLOSURE

FIGURE 18
2 OF 6

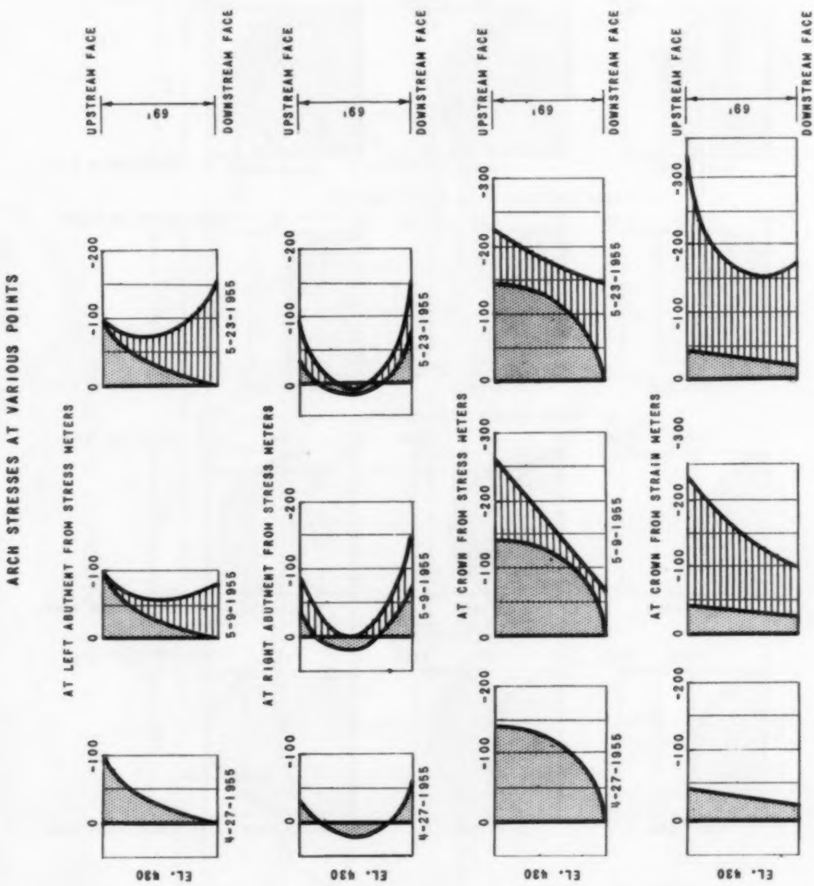


FIGURE 18
3 OF 6

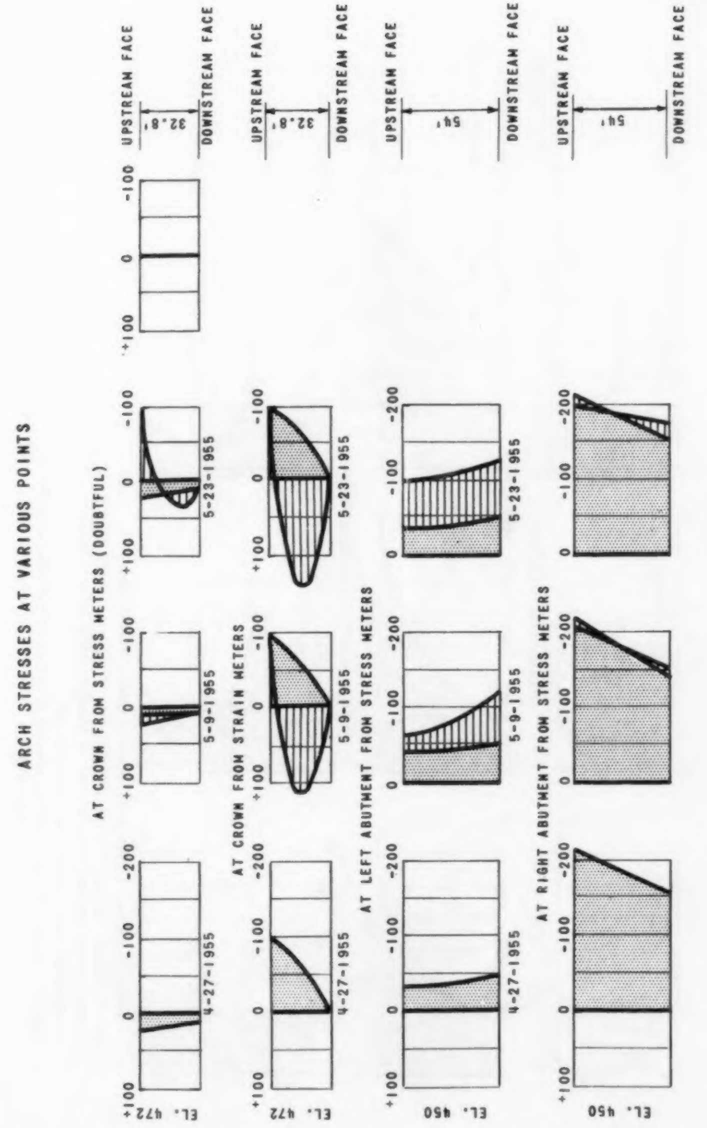


FIGURE 18
4 OF 6

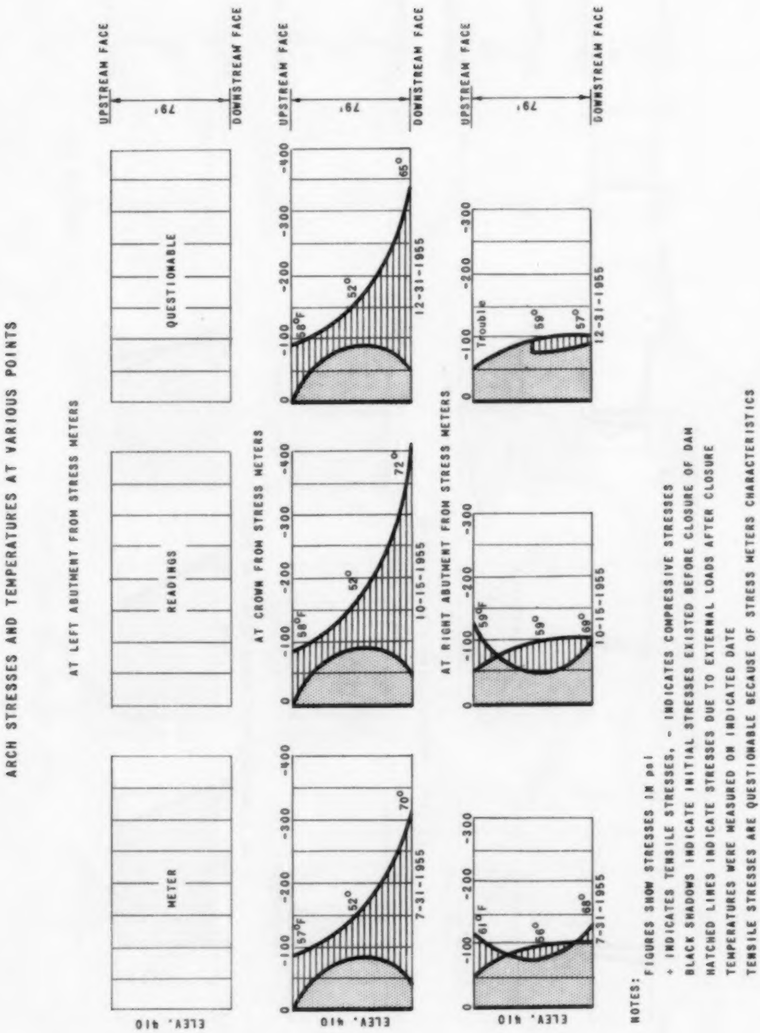


FIGURE 18
5 OF 6

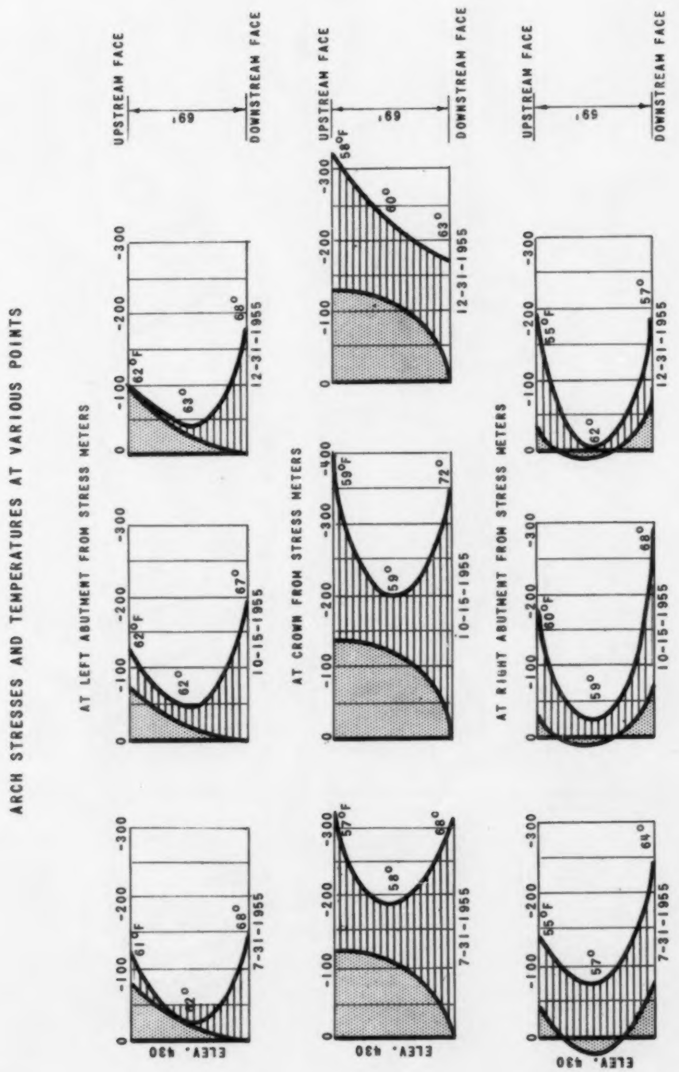


FIGURE 18
6 OF 6

ARCH STRESSES AND TEMPERATURES AT VARIOUS POINTS

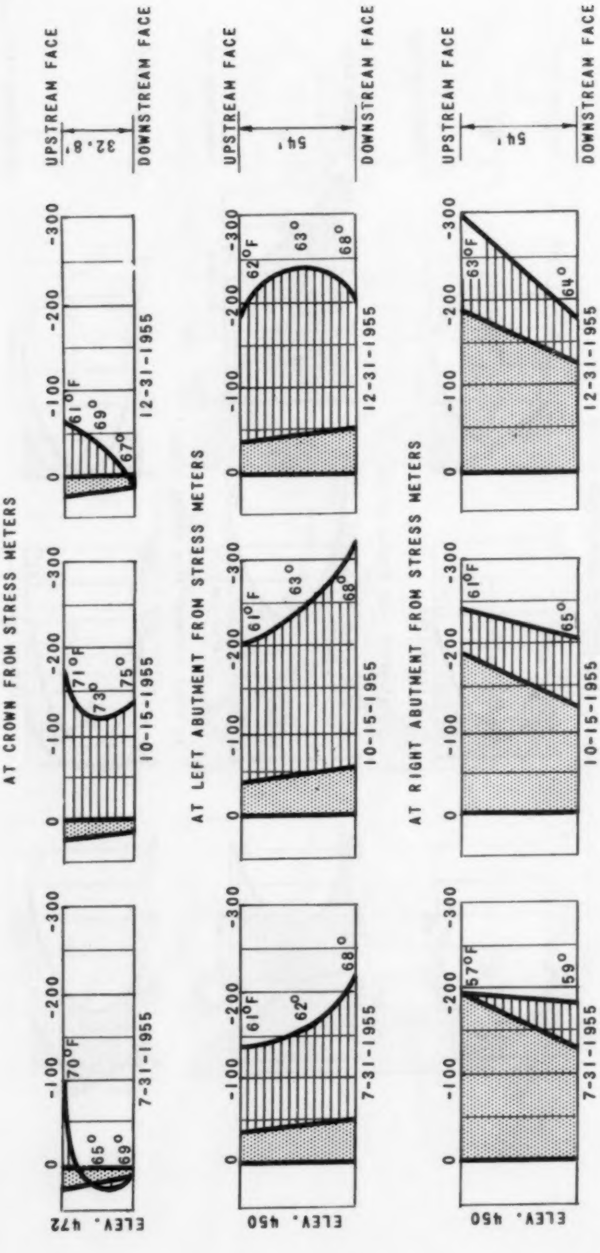


FIGURE 19

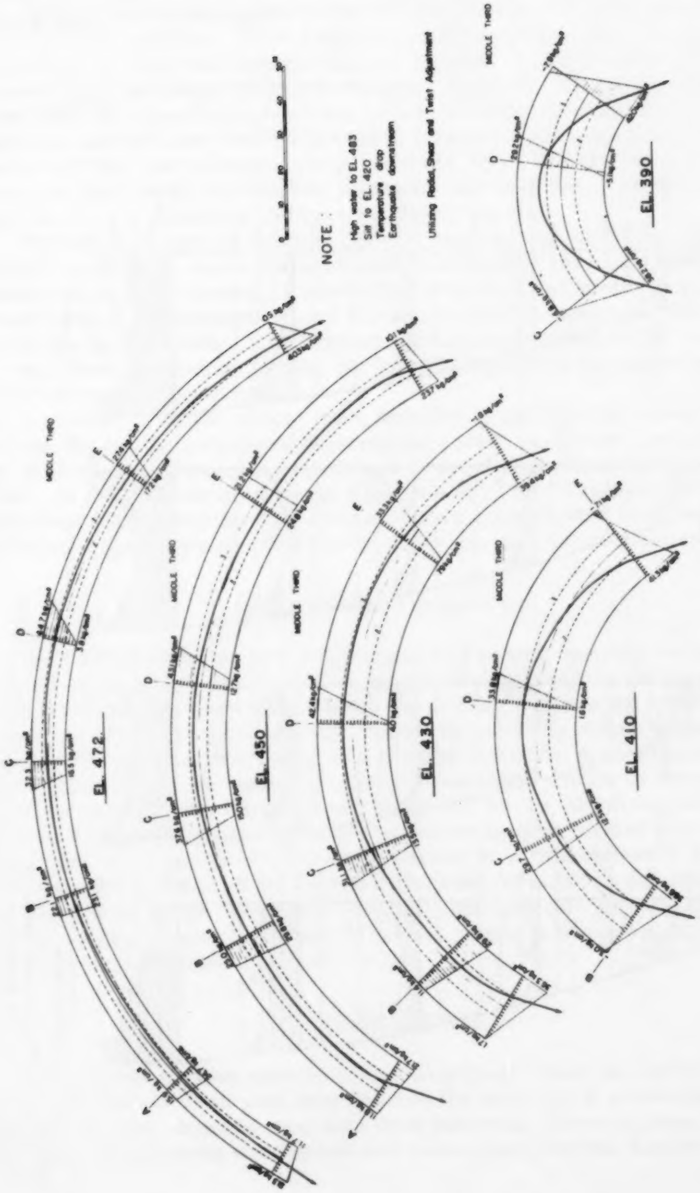
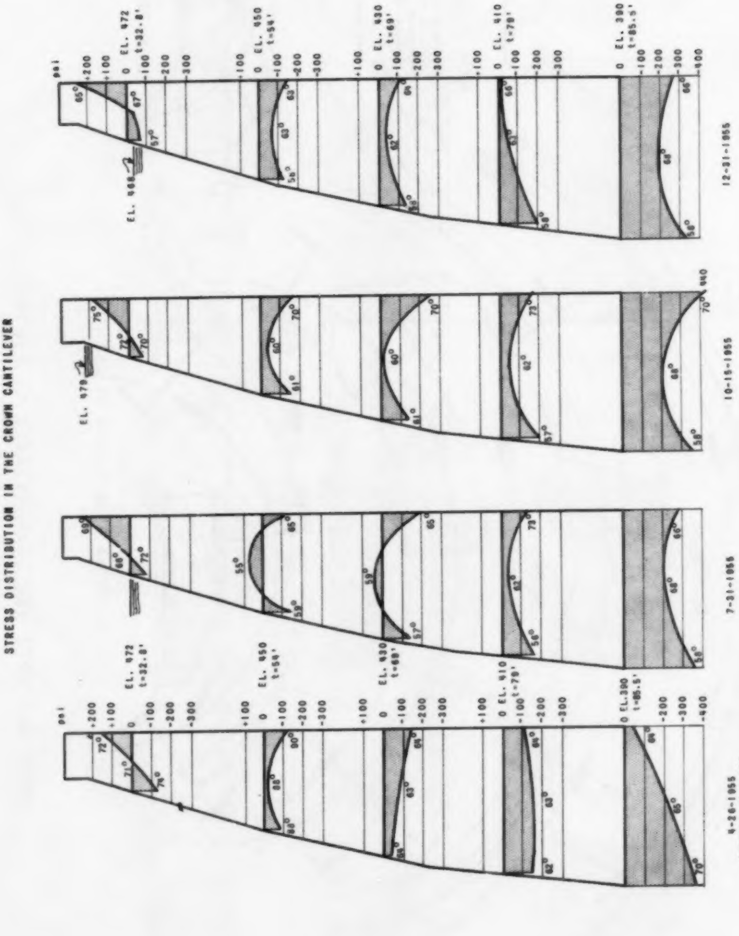

NOTE :
High water to EL 483
Silt to EL. 420
Temperature drop
Earthquake downstream
Utilizing Radial, Shear and Twist Adjustment
EL. 472
EL. 450
EL. 430
EL. 410
EL. 390
MIDDLE THIRD

FIGURE 20


STRESS DISTRIBUTION IN THE CROWN CANTILEVER
4-26-1955
(BEFORE LOADING)
7-31-1955
10-15-1955
12-31-1955
NOTE: - DESIGNATES COMPRESSION
TEMPERATURE SHOWN IN °F

stress distribution is non-linear and is not always consistent with the calculated stresses; tensile stresses calculated by any one of the various methods of computation do not appear in the actual structure in either the crown or the abutment sections. Most important, the figures indicate that the total stresses are less than the calculations indicate. It can be seen from the more recent data that temperature effects exert a greater influence on the arch dam than the effects of water load; and, in addition, that the crown section stresses are greater than the abutment stresses - although the opposite was expected from the calculations. It should be noted that the loading conditions have not completely reached the maximum assumed in the design calculations, and the period of loading has been relatively short.

Figure 20 is included to show the cantilever stress distributions in the crown cantilever. Again it can be seen that the general tendency of the stress distribution is non-linear. It should be mentioned that these forms of stress distributions are frequently seen in mass concrete where temperature distribution is non-linear. The reversed stress distribution at the base of the crown cantilever before loading the dam is believed to be caused by the contraction joint grouting.

A number of strain meters were installed in the right abutment at Elevation 461 for the purpose of determining radial, cantilever, and arch stresses in this area. Figure 21 indicates the variation of stress in this part of the dam. In general, the stresses at Elevation 461 in the cantilever at the right abutment appear similar to the case where a concentrated force has been applied on a semi-infinite elastic body of certain modulus of elasticity.

Stresses Due to Restraint

Each lift of concrete in a structure of this nature receives restraint from the lift immediately below, if their moduli of elasticity are different. A number of meters were embedded in the dam to measure the effects of the restraint of one lift upon another. Figure 22 indicates stress variations immediately following the placing of a lift; this variation is an example where the normal 3-to 4-day time lag existed between the placing of successive lifts. Figure 22 also shows stress variations where the lift below had been placed 50 days before the lift above was placed. (The former were determined by use of stress meters and the latter by strain meters.) These stresses were computed by the use of the several elastic constants previously mentioned. At the location of each of these examples pipe cooling was used. As can be seen on Figure 22 tensile stress is developed approximately 45 to 60 days after placement of the lift.

CONCLUSIONS

Any attempt to draw definite conclusions from either the involved trial load calculations of an arch dam or from the readings of stress and strain meters, at best, depends upon individual judgment. Nevertheless, the authors feel that the following conclusions are warranted from the data presently at hand:

1. An arch dam, properly designed, has an actual factor of safety, as far as stresses are concerned, considerably in excess of the factor of safety indicated by the stress calculations.

FIGURE 21

STRESS DISTRIBUTION AT THE RIGHT ABUTMENT
BLOCK 16 ELEVATION 461, (MEASURED BY STRAIN METERS)

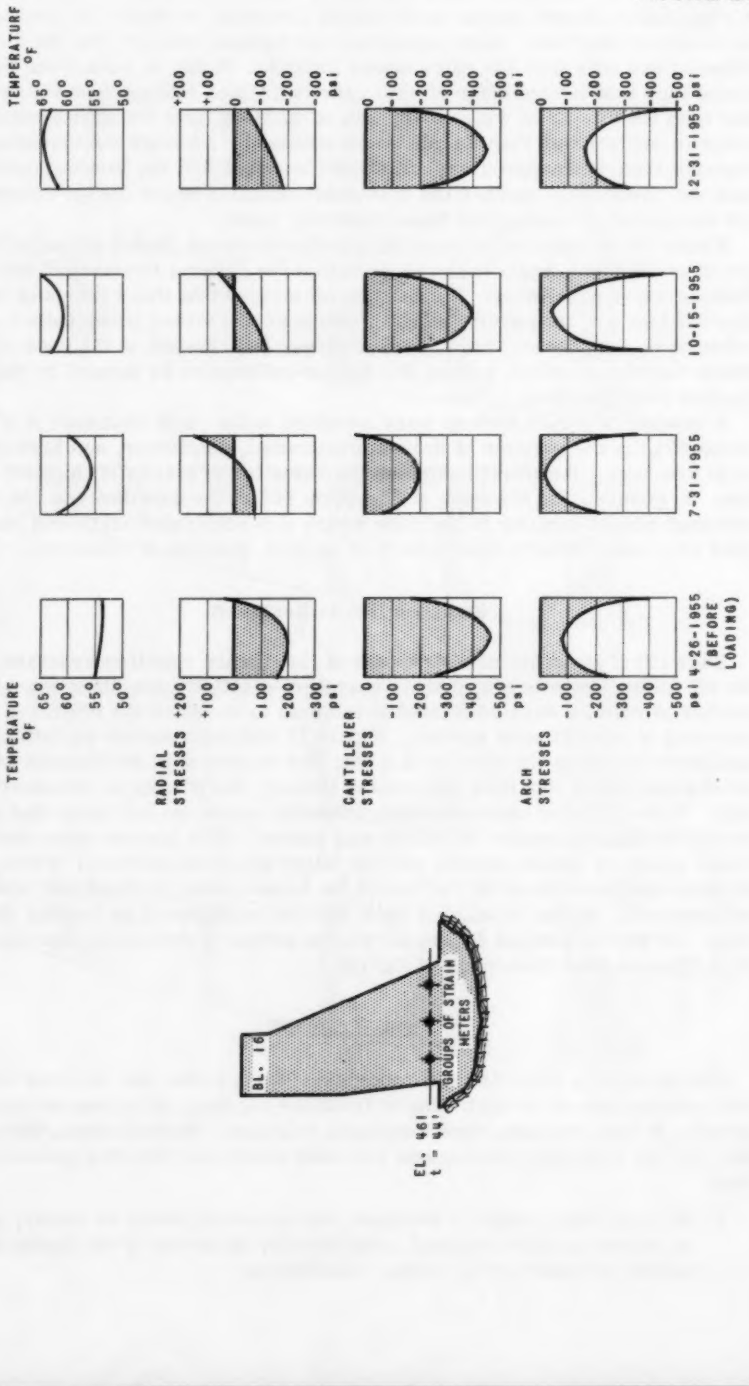
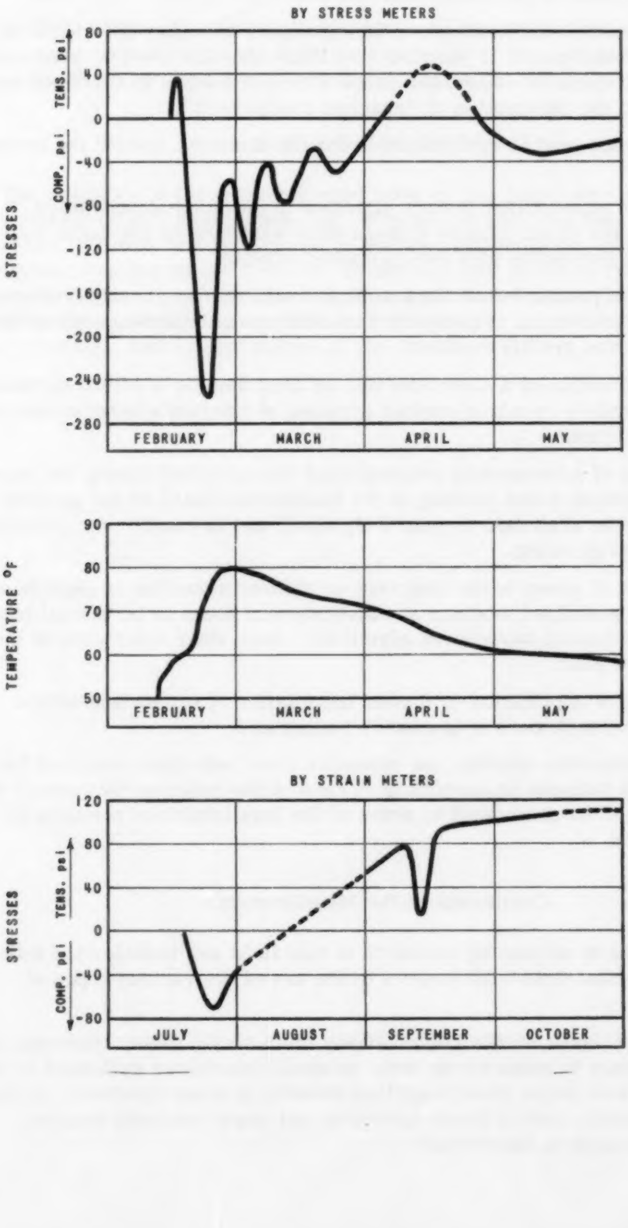


FIGURE 22

STRESSES LARGELY DUE TO RESTRAINT



2. The actual stress distribution in the concrete is at variance with that obtained from the detailed trial-load calculations (this may be said to be especially true of a young structure).
3. Stress distribution is non-linear. It is the temperature distribution rather than the mass temperature of a concrete section that plays an important role in the stress distribution.
4. The calculated tensile stresses do not appear in either the crown or abutment sections; it is possible that these are absorbed by joint opening (which could be called, for calculation purposes, "cracked arches" similar to the assumption of "cracked cantilevers").
5. The stresses tend to concentrate from the abutment toward the crown section.
6. The use of pipe cooling is very effective in reducing temperature stresses; the effect of daily temperature variation on the mass concrete was consistent with the calculated effects.
7. The elastic properties of the foundation rock varied greatly from point to point; however, it is probable that after consolidation grouting this variation was greatly reduced.
8. The construction of a structure like an arch dam on a relatively steep slope definitely requires contact grouting of the joints between the concrete and rock.
9. The effect of a temporary external load (as occurred during the unexpected typhoon water loading of the Kamishiiba Dam) on the grouted portion of an arch dam is relatively small and is easily recovered by subsequent grouting.
10. The effect of creep in the concrete on stress relaxation is significant, since the sustained modulus of elasticity was found to be almost half of the instantaneous modulus of elasticity - even after a duration of loading of one half year.
11. Necessity of duplication or double installation of stress and strain meters to check results is readily recognized.
12. It is questionable whether the extensive time and effort required for a trial-load analysis is justifiable in view of the relative "accuracy" of this method as contrasted to some of the less laborious methods of calculation.

Comments on the Measurements

H. Kimishima is continuing research in this field and includes the following clarifying statements with respect to the use of the various types of meters:

1. In order to make stress computations from strain meter readings, it is necessary to make creep tests on small laboratory cylinders (6 in. or 8 in.) with these results applied directly to mass concrete. Correlation between such a small specimen and mass concrete must be clarified so as to determine:

a. Effect of size of specimen,
b. Effect of maximum size of aggregate, and
c. Effect of adiabatic temperature changes on creep.

2. Poisson's ratio during creep was assumed to be the same as during elastic deformation. This assumption must be checked on mass concrete under the mass curing condition.

3. Until these effects are clarified, the interpretation of stresses from strains is not entirely correct; this leads to the conclusion that stress meters are more reliable than strain meters.

ACKNOWLEDGMENTS

The majority of the work reported upon in this paper was under the direct supervision of Mr. Kimishima. Mr. Kimishima will continue the analyses of stresses, strains, and deflections in the Kamishiiba Dam. It is planned that continuing reports on the behavior of the dam will be made from time to time. The authors are deeply indebted to the many other engineers of the Kyushu Electric Power Company and Ebasco Services who assisted in the installation, readings, and interpretation of the results.

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAM: CONSTRUCTION OF THE KAMISHIIBA ARCH DAM

K. M. Mathisen,[1] and Charles C. Bonin,[2] M. ASCE
(Proc. Paper 1183)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, fifteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); and "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183).

Note: Discussion open until July 1, 1957. Paper 1183 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 1, February, 1957.

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As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

The Kamishiiba Arch Dam was constructed between September, 1952 and June, 1955. In anticipation of the adoption of either an arch or gravity dam, the diversion tunnel and upstream cofferdam had been completed prior to September, 1952.

Construction engineers from Ebasco Services Incorporated were assigned to assist engineers of the Kyushu Electric Power Company in the construction of the arch dam. It was the main task of the American engineers to assist in the preparation of engineering and design specifications and to confirm that these were being met by the construction forces.

As a result of many unforeseen factors, the actual construction period was 30 months although this was 6 months more than actually required for the job. Delays were caused by shortages of units of the construction equipment, tools, and spare parts, and by damages caused by two severe typhoons in the autumn of 1954 to portions of the construction plant and partially completed project.

A token placement of concrete was made in the dam on June 2, 1953; major concreting started in November, 1953. The diversion tunnel was closed during January, 1955; the construction sluiceway through the arch dam was closed on April 27, 1955, at which time storage of water in the reservoir began. On May 26, 1955, the powerhouse was put into operation.

Plant and Equipment

Assembling the equipment and erecting the construction plant was begun in September, 1952. This was to be the largest and most modern plant that had ever been used for heavy construction in Japan. The plant and most of the large equipment were owned by the power company and used by the contractor and subcontractors employed to build the dam. The contractor supplied some of the equipment and was responsible for all equipment and plant maintenance.

The plant was arranged on the right side of the river valley as indicated on Plate 1. It was dominated by the crushing and screening plant which was capable of producing 300 tons of crushed rock and sand per hour. The rock was graded into 16, 8, 4 and 2 centimeter sizes. Primary crushing was done by two 150-hp jaw crushers; two 100-hp gyratory crushers and three 100-hp cone crushers were employed for secondary crushing. Sand was manufactured by three 35-ton-per-hour rod mills. The sand and aggregate were delivered to the concrete batching and mixing plant by belt conveyors which passed the aggregate through a final washing and dewatering station.

An important feature of the plant was the 57-kilometer aerial cableway used for delivering cement, sea sand for concrete for other portions of the development and miscellaneous materials. The cableway had been built

originally to supply sand and cement for the construction of two existing downstream developments; the cableway was rehabilitated and extended to serve this project. It was capable of delivering materials, packed in its 9-cu-ft buckets, at the rate of 40 tons per hour. Approximately 130,000 tons of the cement used for the development were delivered by this cableway.

The cement for the dam was delivered to a warehouse at the cableway terminal where the cement was unsacked and pneumatically transported to two 750-ton storage silos. From these, it was delivered by a 40-ton-per-hour capacity system of conveyors to the concrete plant. The concrete plant was capable of producing 120 cubic meters per hour in its three 4-cu-yd mixers. The concrete plant was fully automatic and designed to be operated by one man. Laboratory facilities were installed for the concrete control inspectors.

Water for construction uses was obtained from a small tributary near the dam site. The pumping station contained seven multiple-stage centrifugal pumps capable of supplying 13 cubic meters of water per minute to the main storage tank. Located adjacent to the batch plant was the refrigeration and ice plant capable of cooling 1800 liters of water per hour to 1.6^{o} C. This cooled water was used in the concrete mix and for cooling by circulation through embedded cooling coils in the dam. The ice plant was also used for producing up to 30 tons of ice per day that could be substituted for a part of the concrete mixing water.

All concrete, except small quantities used for miscellaneous construction, was delivered from the concrete plant to the placing site in single chamber 4.5-cubic-meter buckets. A double-track transfer railway was employed to move the empty and full buckets between the plant and the pickup zone of the construction cableways. Two 13.5-ton capacity flatcars and three 81-hp diesel locomotives were used in this service. The buckets were operated by applying compressed air to the dumping mechanism at the placing site.

Twin 13.5-metric-ton capacity cableways, with a span of 430 meters between towers, served the dam site. Each was capable of delivering concrete to any location in the dam at approximately 60 cubic meters per hour. They had a common fixed head tower on the right side of the valley and individual movable tail towers mounted on a curved tail-tower track located above the left abutment and thrust block. Their carriages traveled at speeds up to 360 meters per minute, and their load lifting speed was as high as 90 meters per minute. Vertical and horizontal motions were independent; the mechanism was not designed for both to be operated simultaneously. The tail towers traversed upstream or downstream at 12.5 meters per minute in a separate motion or simultaneously with either one of the other two motions.

Other features of the plant included the necessary warehouses and shops, and included a double-track funicular railway which was used to haul aggregate rock salvaged from excavation operations from the river bottom to the primary crusher. It had two 7.7-ton capacity skips that were alternately loaded and dumped giving the machine a maximum capacity of 75 tons per hour.

Two small bulldozers were delivered to Kamishiiba in the summer of 1951. A few other pieces of equipment followed at odd intervals until early in 1953 when the bulk of the equipment arrived. After this, additional equipment was delivered at scattered intervals until the end of 1954. The principal units of equipment used in the construction of the dam are listed below:

2 American-built 1-3/4-cu-yd diesel-power shovels
1 American-built 3/4-cu-yd diesel-power shovel
3 Japanese-built 1.2-cubic-meter diesel-power shovels
1 Japanese-built 1.5-cubic-meter electric-power shovel
1 European-built 2.0-cu-yd diesel-power shovel
6 Japanese-built D-50, 44-hp bulldozers
4 Japanese-built D-80, 75-hp bulldozers
6 American-built D-8, 1113-hp bulldozers
2 American-built D-7, 80-hp bulldozers
1 Japanese-built 75-hp road grader (removed in 1953)
9 American-built 10-cu-yd end dump trucks
13 Japanese-built 4-cu-yd end dump trucks
16 American-built wagon drills
10 Japanese-built wagon drills
73 Japanese-built jack hammers
17 Rotary drilling machines
5 Japanese-built grout pumps with mixers
2 American-built grout pumps with mixers
3 American-built 600-cu-ft-per-minute portable air compressors
55 Japanese-built concrete vibrators
10 American-built concrete vibrators
7 Japanese-built portable electric-arc welders
5 Japanese-built portable acetylene welders
Several Japanese-built 2-ton trucks used for hauling materials and supplies

Most of the pieces of lesser equipment, not listed, were of Japanese manufacture. The concrete plant and most of the refrigeration and ice plant machinery were the largest units of the construction plant that were imported into Japan. The remainder of the plant was of Japanese manufacture or built by Japanese firms under license from American and European manufacturers.

Foundation Excavation

The location of the arch dam was governed by the foundation rock at the site; alignment was not symmetrical with the river channel and valley. This condition, plus the fact that a thrust block and cutoff wall were necessary on the left abutment, required that a greater volume of rock be excavated on the left abutment than on the right abutment. It was also necessary to do some excavation in the left forebay to provide an adequate channel to the spillway in block #4 (see Plate 2). The depth of excavation at any elevation was governed by the requirements that the rock be sound and adequately strong, the excavated foundation face be radial, and the slope of the abutment be continuous with no sharp breaks or wide benches.

Removing and wasting the excavated material presented a difficult problem in the confined river valley. It had been planned that all acceptable rock be delivered to the crushing plant to be made into aggregate. This was never fully accomplished because of the late installation of the funicular railway for hauling the rock, and the failure to segregate and stockpile the acceptable rock. Therefore, most of the material was eventually wasted in a spoil area on the right river bank about one kilometer below the dam. Little material was wasted in the reservoir area because there were no roads on which to

operate equipment on either side of the valley above the dam.

The foundations for blocks 11, 12 and 13 on the right abutment were completed first. The second area completed was the river channel in which blocks 7, 8, 9 and 10 were located. Thus, difficult excavation remained to be done on both abutments above concrete-placing operations in the completed areas. At times these operations conflicted and required delicate scheduling in order not to result in excessive delays. Excavation of the upper right abutment was complicated by the crossing of the transfer railway and sometimes delayed by slides into the abutment area above the railway.

In March 1955 foundation excavation was completed. In spite of the tedious manner in which the foundation was excavated (because of the necessity of using local methods and equipment), the foundation rock was prepared in a thorough and acceptable manner. The rock exposed on the right abutment was jointed and weathered deeper than anticipated. In the river bottom, on the left abutment and especially in the thrust block area the foundation rock was better than anticipated. The final profile varied only slightly from the assumed profile for the final design. Approximately 290,000 cubic meters of rock and overburden were excavated to provide a sound foundation for the arch dam, thrust block and cutoff, and spillway chutes.

Foundation Grouting

In general, foundation "consolidation grouting" was begun in an area only after it had been satisfactorily excavated and accepted for construction. "Curtain grouting" was considered the second phase of the foundation grouting program.

Consolidation grouting was started in blocks 12 and 13 in May 1953. Eight-, 12- and 16-meter deep holes were drilled on 5-meter centers in a rectangular pattern. Those holes 8-meters deep were drilled with 3-in. wagon drills. The others were 60-millimeter shot or diamond drill holes. The holes were washed with an air-water jet pipe, then threaded 2-in. pipe nipples were set in the holes; pressure testing and grouting were generally done at 50 psi.

The results of this early work showed that the planned grouting program required revision. A new hole pattern was adopted that was a compromise between grouting before placing any concrete and allowing concrete placing to begin without delay. The revised program planned alternate rows of 3- and 7-meter deep holes across the foundation. The rows were 2-1/2 meters apart, and the holes were drilled normal to the general slope of the foundation, 5 meters apart in the rows. The holes in one row were offset 2-1/2 meters from the holes in the adjacent rows. Drilling, washing, testing and grouting the rows of holes proceeded in an uphill direction ahead of placing the lifts of concrete, or were done between placing lifts of concrete from the top of the concrete in place. The latter procedure reduced troublesome surface leakage that often interfered with the work. In some areas all the holes were drilled 7 meters deep. When this pattern was completed in a block, two, four or six holes were drilled 16 meters deep. These holes were piped to the downstream face of the dam, to be grouted after concrete was placed in the block to a height of 16 meters or more above the foundation.

A system was developed to control the grout mix and the grouting pressure. This was based on the rate of grout take during a specified period of

time. The mixes varied from a W/C ratio of 10 to a W/C ratio of 1 by weight. The pressure ranges were:

3 meter holes - 15-psi start to 25-psi finish
7 meter holes - 25-psi start to 50-psi finish
16 meter holes - 50-psi start to 75-psi finish

Additional holes were drilled and grouted in the area of any hole that took over five sacks of cement. This pattern and procedure were used over the entire foundation. The total drilling for consolidation grouting was 7300 meters.

A brief summary of the consolidation grouting is as follows:

	Average Take: 50-Kg Sacks Per Meter		
Block	3-Meter	7-Meter	16-Meter
Thrust B1, 2 and 3	0.19	0.14	0.44
4, 5, 6 and 7	0.23	0.25	0.95
8, 9 and 10	0.25	0.51	0.85
11, 12, 13 and 14	0.52	0.61	2.59
15, 16 and 17	0.13	0.33	0.92
TOTAL	0.34	0.42	1.32

These "takes" reflect the quality of the foundation in the various areas; the rock under blocks 11, 12, 13 and 14 possessed the most seams and fissures, while that under the thrust block and blocks 2 and 3 was the soundest.

A continuous grout curtain was produced by testing and grouting a row of holes drilled into the rock beneath the dam. The curtain was extended a short distance along the reservoir rim beyond each end of the dam. Curtain grouting was started early in 1953 on the right reservoir rim and was completed in July 1955 on the left reservoir rim. Approximately 7000 meters of hole were drilled and grouted to complete the rim curtain work. It was done entirely by the stage drilling and grouting method. Stage grouting using deep packers was considered, but rejected because it was believed this would not be as satisfactory with the type of worker available at the site.

Curtain grouting under the dam was begun in January 1954 in block 9 in the river channel. From this point the grouting was carried upward on both abutments as the blocks were completed to heights approximately 30 meters above their foundations. All work was done in this order:

1) "Exploratory" holes, 50 meters deep, 20 meters center to center were drilled in three equal length stages that were grouted at 100-, 200- and 300-psi pressures.
2) "First intermediate" holes, located halfway between "exploratory" holes, were started when the first two stages of the adjacent holes were completed. They were drilled one-third of the waterhead but not less than 25 meters deep. They were drilled in two equal length stages and grouted at 100- and 200-psi pressures.
3) "Second intermediate" and "third intermediate" holes were similar to and followed the "first intermediate" holes in order. They were located halfway between adjacent existing holes making the final hole spacing 2-1/2 meters center to center.

The curtain hole spacing was spread to 3-1/3 meters center to center in

blocks 5 and 6, and to 5 meters in block 4. Several diagonal check holes were required on the right abutment in zones of relatively high take and to check-grout several small vertical seams.

Most of the earlier drilling was done using 65-millimeter shot drills. This drilling was very slow; the advance of the bit being controlled entirely by the force exerted on the machine by the driller. It was used for reasons of economy, diamond bits being very expensive and labor costs being relatively low. Diamond drilling, using "EX" size bits eventually replaced the shot drilling; at the conclusion of the curtain grouting program, it was the only method being used. The curtain grouting has proven very satisfactory; seepage under the dam has been negligible.

The grout takes for the curtain were:

Exploratory Holes	1.5	50-kg sacks per meter
First Intermediate Holes	0.6	50-kg sacks per meter
Second Intermediate Holes	0.4	50-kg sacks per meter
Third Intermediate Holes	0.4	50-kg sacks per meter
Right Abutment Check Holes	0.3	50-kg sacks per meter

Quarry Operations and Aggregate Production

Shortages of aggregate and sand caused several minor delays in the construction of the dam. The problem was primarily one of quarry operation, and secondarily one of scheduling the heavy earth-moving equipment between dam-site excavation and the quarry. The area selected as a quarry contained quantities of sound graywacke adequate for the concrete required in the dam, but it also contained thick beds of slate and zones of deep weathering. Coyote-hole blasting was used in the quarry; this mixed quantities of the unacceptable rock and some overburden with the good rock and also left a number of extra large pieces which slightly delayed operations by requiring drilling and secondary blasting.

Between July 1953 and July 1954, 81,200 tons of sound rock were salvaged from the power tunnel-excavation muck. This was 9% of the aggregate rock hauled to the crusher. Between October 1953 and March 1955, 181,700 tons of rock were salvaged from the foundation excavation and from river-channel improvement excavation at the end of the spillway chutes. This accounted for 19% of the aggregate rock. A total of 980,000 tons of broken rock were delivered to the crushing plant for crushed-rock aggregate and manufactured sand for concrete production.

Various means were employed to assure that the highest quality of aggregate was used in the concrete. Washing stations were set up before and after the primary crusher. Pickers were stationed along the various conveyor belts to remove by hand pieces of unacceptable aggregate. Rock from the three sources was blended to produce the best possible aggregate from the material delivered to the crusher.

Concrete Control

The design of concrete mixes, and the investigation and testing of available brands and types of cement, was begun in October 1952. Concrete specifications required a minimum of 235 kg of cement per cubic meter of concrete

placed in contact with the foundation, placed within one meter of the faces of the dam, or placed in reinforced sections. This was designated "A" mix. A minimum of 210 kg of cement per cubic meter was specified for interior mass concrete for the dam, designated "B" mix. In 1954, a "C" mix containing 190 kg of cement per cubic meter was adopted for miscellaneous concrete work. Type II cement as described in U.S. Federal Specifications SS-C-192 was adopted for all cement in the dam.

A thorough concrete and aggregate testing program was established. Inspectors were stationed at the quarry, crushing plant, concrete plant and the placing site. The gradation and moisture of the aggregate and sand were checked at least once each shift. Slump tests were made and the entrained air content was determined at frequent intervals. Standard 6" x 12" test cylinders were cast during each shift for 7, 28 and 91 days, 6 months' and 1-year break tests. Testing of the cement was carried out in the manufacturers' laboratories and periodically witnessed by engineers from the field laboratory. Overall quality control was entirely adequate and remarkably effective.

The monthly average of concrete strengths obtained at 91 days are given in Table 1.

Construction Operations

The dam, as constructed, is shown in plan and profile on Plates 2 and 3. The arch blocks, thrust block and cutoff were constructed with two-meter concrete lifts, except on the foundation where one-meter lifts were specified.

Contact grouting systems were installed on all foundation surfaces having a general slope steeper than 30 degrees from the horizontal. Each system contained two horizontal 1-1/2-in. header pipes joined by 3/4-in. riser tubes to which the necessary grout outlets were attached. All headers opened at the downstream side of the dam.

Timber forms were utilized for all concrete placed. Most of the lumber and timber required was cut by sawmills at the job site. Once off the foundation, raisable form panels were used at the faces of the blocks. These panels had vertical sheathing planks and were sufficiently flexible to be easily warped to the curvature of the arch. Two-inch chamfer strips were used at the horizontal construction joints and the vertical block joints. The joint forms of the high blocks consisted of a series of keyway forms connected by narrow panels backed by timber whalers. All forms were anchored by embedded metal tie rods. The vertical water and grout stops, the horizontal grout stops, the grout boxes and tubing systems and the grout vent grooves were all fastened to the joint forms.

The concrete placing procedure was to begin at the downstream face of a block and finish at the upstream face. All concrete was placed in terraces about one-half meter thick and thoroughly vibrated into place. One cableway was used per block, except in the large base lift of the blocks in the river bottom, where both cableways were used and placing was started at both faces of the dam to be completed at the center of the block. The surface of each lift was green-cut using jets of air and water after the concrete had taken its final set. It was often necessary to start this operation on the downstream portion of a lift while concrete was still being placed in the upstream portion. After green-cutting, and during forming for the next lift, the cooling

coils were installed on the concrete surface. The 1-in. thin wall steel tubes were spaced at 1, 1-1/2 and 2 meters depending upon the location in the block and in the dam. They were fastened by means of wires set in the surfaces of the concrete lifts. Slip-type tubular couplings were used to fasten the lengths of tubes and the elbows together.

A cooling plan was set up in which refrigerated water was circulated through the embedded coils for a primary cooling period of approximately 14 days. After several weeks a secondary cooling was initiated. This was designed to cool the concrete to the mean ambient temperature of 15.6° C. Cold river water was utilized during the winter months. The cooling program was remarkably successful in preventing unduly high concrete temperatures and in preventing cracks in the completed structure. Cooling was usually continuous until the concrete was at or near the mean ambient temperature.

The capacity to produce and place concrete was entirely adequate for the job. With the opening of the river bottom in November 1953, the concrete placement began in earnest, and continued on a normal schedule for 14 months until the project was nearly complete. In January 1954, slightly over 2400 cubic meters were placed in just less than 24 hours; this established a new placing record in Japan at that time. After this, hourly rates often exceeded 100 cubic meters per hour. The total volume of concrete as measured at the batch plant was 381,357.5 cubic meters. This volume was placed in the dam, thrust block, cutoff and spillways.

Contraction Joint Grouting

In order to obtain a monolithic structure, it was necessary to thoroughly grout the vertical contraction joints. These joints were divided into 20-meter grout lifts. Each lift was enclosed by Z-shaped copper seals anchored in the concrete of the adjacent blocks. Systems of embedded tubing with grout outlet boxes in the joint were installed to inject the grout into the joints. The outlet boxes were spaced 3 meters vertically and the riser tubes 2 meters apart. The vent groove at the top of each lift was not connected to the tubing system.

The grout lifts were established by placing horizontal grout stops at the foundation rock and at elevations 390, 410, 430, 450, 472 and 482 meters. In the spring of 1954, the elevation 430 to 450 grout lift was divided into two portions by adding a stop at elevation 440. This permitted joint grouting as high as possible before closing the dam in September, as was then scheduled. Reinjectable grout systems imported from France were installed in addition to the regular systems above elevation 460 and were the only systems installed above elevation 472.

In the spring of 1954, grouting of concrete-rock contact systems installed on the lower portions of the foundation was begun. These systems were washed for several hours, pressure tested at 1 psi for each 1-ft height of concrete above the system, then grouted at the same pressure using a grout mix with a W/C ratio of 4 by volume, or thicker when it was desirable. Grouting was continued until refusal; pressure was maintained on the system for two more hours. Water was circulated in adjacent ungrouted contact and joint grouting systems to protect them from damage from plugging by grout intrusions from the system being grouted.

All contact systems located below the elevation of the top of a grout lift were completed before joint grouting was started in that lift. All contact grouting was delayed until the concrete had cooled to 15.6° C. Grout takes were generally small, although a few systems had relatively high takes; these latter were usually on the steeper foundation surfaces. A summary of the contact grouting is shown on Table 2.

The contraction joint grouting, begun in May 1954, was the first such program in Japan. The majority of the lifts of the joints were grouted individually; there were cases where it was necessary or desirable to grout two joints in one lift or two lifts in one joint simultaneously. All joints were satisfactorily grouted up to elevation 472 by the end of April 1955. Above elevation 460, concrete temperatures were above the mean ambient temperature of 15.6° C when the joints were grouted. The reinjectable systems were installed for just such a situation and were used in the spring of 1956 for regrouting.

In scheduling the joint grouting, temperature of the concrete was the prime consideration. Mean ambient temperature or lower was considered necessary for maximum concrete shrinkage, maximum joint opening and maximum effectiveness of grouting. At times it was necessary to grout with concrete temperatures nearly one degree higher. Temperatures were determined by the embedded thermometers, stress meters, strain meters and joint meters. These readings were intermittently checked by filling selected cooling coils with water and capping them for as long as 48 hours. The water was then slowly forced out as its temperature was measured. The average temperature of such water corresponded very closely with the temperature readings of the embedded instruments.

Once a lift was ready for grouting, the joints were given a preliminary water test to locate any serious leaks. Leaks of some magnitude existed in nearly every joint; all face leaks were caulked before grouting began. Specifications required that adjacent ungrouted joints be filled with water and the headers capped; this was done to minimize block movement and prevent the adjacent joints from being closed by grouting pressures. Experience showed that few of the joints were sufficiently watertight to accomplish this; therefore, it was necessary to use flowing water in the adjacent ungrouted joints. Flowing water was also maintained in the lift of the joint above the lift being grouted. In cases of known open leaks, this water was maintained under enough pressure to eliminate the leakage past the horizontal grout stop.

During all water testing and grouting operations, dial gages were mounted on the downstream face of the dam across the joint or joints being tested or grouted, and also placed across those adjacent joints in which water was circulated. Testing and grouting were done at pressures at the upper vent headers that produced a maximum movement of 0.8 millimeter or at a maximum of 60 psi.

Grouting was done by following a routine procedure. The grout was introduced to the lower supply header, circulated through it and returned to the mixer. The return header valve was then closed and the grout forced into and upward through the joint and out the open upper vent header valves. These valves were then closed and the pressure in the joint controlled by properly throttling the lower return header valve. The upper vent header valves were cracked as often as necessary to bleed off excess water and thin grout. Grout was circulated continuously through the lower supply-return header to replace the water lost through bleeding. This was continued until

the joint was completely grouted. Pressure was then held on the joint for an additional two hours to allow the grout to set.

Type I cement was used for all joint grouting. To remove lumps and any possible foreign matter the cement was screened through a #30 screen immediately before use. The W/C ratio of the grout mix used was 1 by volume. The initial batch of each operation usually had a W/C ratio of 2 or 4 by volume. The initial joint openings as indicated by the embedded joint meters ranged from a minimum of 0.5 millimeter to a maximum of slightly over 4 millimeters.

After completion of this work, several core holes were drilled into joints which had been grouted to sample the grout deposits. In general, these check holes revealed that the joint grouting had been very effective.

Table 3 is a tabulation summarizing the contraction joint grouting.

Closure of Dam

The plan for closure of the dam was to follow a sequence as follows:

1) A vertical shaft, approximately 1-1/2 meters in diameter, had been excavated from the heel of block 4 to the location chosen for the concrete plug in the diversion tunnel.
2) Two holes had been cut in the concrete arch cofferdam. Timber flap gates to cover the holes were installed on the upstream face of the cofferdam and secured in the open position.
3) The diversion tunnel was closed by setting stop logs in the intake portal of the tunnel thus diverting the river through the openings in the cofferdam and into the construction sluiceway through the arch dam.
4) After the diversion tunnel completely drained, keyways were excavated in the tunnel walls for a plug 10 meters in length, the plug area cleaned, and radial grout holes drilled for grouting after the plug was completed.
5) Concrete for the tunnel plug was placed by using spouts suspended down the shaft; two sets of cooling coils were embedded in the plug; the vertical shaft was completely backfilled with concrete.
6) After the plug was completed and grouted the flap gates were dropped closing the holes in the cofferdam. This allowed a short period of time (approximately 3 hours) for the water to drain through the sluiceway, exposing the closure gate seats. The seats were cleaned and the gate lowered into place sealing the sluiceway.
7) The sluiceway was then backfilled with concrete and the plug sealed by grouting.

By March 1955, the first five steps of the closure plan had been completed and all joints up to elevation 450 had been grouted. River discharge was low, making this an opportune time to complete closure of the dam. However, in view of the possibility of early spring floods raising reservoir levels quickly to the spillway crest, decision was reached that all joints should be grouted up to elevation 472 and that the curtain grouting should be completed under all portions of the dam before closing. Allowing time for this work, as well as for other operations, April 16th was set as the closure date. Closure was delayed by 8-1/2 in. of rain and a small flood which occurred April 15. On

April 27, 1955, closure was accomplished. One flap gate temporarily failed to close, which caused momentary difficulty and anxiety; nevertheless, the sluiceway gate was successfully sealed, but just 2 minutes before the cofferdam was overtopped.

With the closure of the dam, reservoir level rose quickly and on May 26, 1955 reached a level that permitted the power plant to go into operation. Later, on June 25, 1955, the Kamishiiba Hydro-Electric Development was formally dedicated. And so, after 30 months of hard work, in spite of extremely difficult topographic conditions and the adversities of nature, the combined efforts of Japanese and American engineers were able to successfully complete the first arch dam to be constructed in the Far East.

TABLE 1

AVERAGE 91-DAY COMPRESSIVE STRENGTHS OF 6" x 12" TEST CYLINDERS

(Kg/cm^2)

Period	"A" Mix	"B" Mix	"C" Mix
June-October 1953	385	333	-
November	397	358	-
December 1953	390	360	-
January 1954	375	347	-
February	406	350	-
March	435	384	347
April	375	373	281
May	410	345	304
June	400	-	318
July	410*	390	321
August	388	357	312
September	390*	355	287
October	412	389	337
November	444	-	363
December 1954	440*	427	338

* Approximate

TABLE 2

SUMMARY OF CONTACT GROUTING

Operation Number	Location	Foundation Slope	Area M^2	Outlets	Area/ Outlet	Estimated Net Take 50 kg Sacks
1	Bl 7	40^o	390	39	10.0	1.4
2	Bl 10 Lower	35^o	155	32	4.8	2.3
3	Bl 8-9	25^o	55	11	5.0	1.8
4	Bl 10 Upper	45^o	380	54	7.0	13.5
5	Bl 9	40^o	160	25	6.4	2.0
6	Bl 11 Lower	40^o	100	16	6.2	0.3
7	Bl 11 Upper	35^o	220	23	9.6	7.6
8	Bl 6 Lower	35^o	75	23	3.2	2.9
9	Bl 6 Upper	40^o	340	24	14.1	8.7
10	Bl 12	90^o	90	28	3.2	10.0
11	Bl 13	45^o	320	37	8.7	1.1
12	Bl 5	45^o	204	30	6.8	1.4
13	Bl 3	55^o	238	37	6.4	5.2
14	Bl 15	55^o	140	28	5.0	2.9
15	Bl 2	50^o	100	18	5.5	9.6
16	Bl 1	40^o	210	22	9.5	1.1
17	Bl 16-17	90^o	30	6	5.0	85.1*
18	Diversion Tunnel Plug - Take not determined					

* System in end of old foundation exploration tunnel

TABLE 3

SUMMARY OF CONTRACTION JOINT GROUTING

Grouting Operation	Joint	(Elev. in Meters) Lift	Sq. Meters Area	Grouting Pressure psi At Top of Lift	Opening Movement Recorded mm 1/	Est. Avg. Joint Opening Under Grouting Pressure mm	Estimated Cement Required 50-kg Sacks	Total Cement Accounted 50-kg Sacks 2/	Liters/Min. Leakage Indicated by Water Pressure Test
1	9-10	Rock-390	362	60	0.4	1.4	20.4	41	Trace
2	(8-9	Rock-390	448	60	)	1.3	65.8	163	23
	(8-9	390-410	642	35	0.7)				
3	7-8	Rock-390	250	60	0.1	0.8	7.2	22	13
4	(6-7	Rock-410	413	60	0.5	2.2)	76.1	157	(30
	(7-8	390-410	532	60	0.7	1.6)			(60
5	(10-11	Rock-410	402	60	0.2)	1.3	19.4	39	12
	(11-12	Rock-410	90	60	)				2
6	9-10	390-410	542	60	0.5	1.4	34.6	70	
7	9-10	410-430	487	40	0.8	2.1	43.3	239	75
8	7-8	410-430	473	30	0.7	2.2	43.9	164	110
9	8-9	410-430	486	50	0.5	1.9	39.7	97	47
10	10-11	410-430	483	60	0.6	1.8	37.8	25	16
11	6-7	410-430	484	40	0.8	2.5	50.1	111	
12	(11-12	410-430	483	60	0.0	1.1)	31.2	46	10
	(12-13	Rock-430	233	60	0.0	0.3)			10
13	5-6	Rock-430	413	60	0.4	2.2	38.3	130	26
14	4-5	Rock-430	99	60	0.2	2.0	8.4	26	58
15	8-9	430-440	204	60	0.3	2.5	21.0	20	8
16	(10-11	430-440	204	60	0.6	1.7)	28.3	53	(8
	(11-12	430-440	204	60	0.3	1.4)			(8
17	6-7	430-440	204	60	0.8	3.4	31.3	50	12
18	5-6	430-440	204	60	0.7	2.7	22.5	32	36
19	7-8	430-440	204	60	0.5	2.9	24.0	35	9
20	9-10	430-440	204	55	0.4	2.5	21.1	40	25
21	(12-13	430-442	233	60	0.3	1.0)	21.2	32	(6
	(13-14	Rock-442	172	60	0.3	1.2)			(18
22	3-4	Rock-440	178	60	0.6	2.3	17.1	24	8
23	4-5	430-440	204	60	0.4	2.2	18.9	146	47
24	8-9	440-450	178	60	0.3	4.0	28.0	23	5
25	9-10	440-450	178	50	0.4	3.0	21.6	40	3
26	11-12	440-450	178	60	0.4	1.7	13.3	37	8
27	6-7	440-450	178	50	0.8	4.7	32.5	58	6
28	5-6	440-450	178	50	0.6	3.6	25.5	59	31
29	13-14	442-450	132	60	0.5	1.6	9.4	16	12
30	14-15	Rock-450	188	60	0.3	1.5	12.7	20	11
31	3-4	440-450	178	60	0.4	2.6	19.1	21	12
32	2-3	Rock-450	58	60	0.4	2.2	5.4	14	6
33	4-5	440-450	178	35	0.6	2.7	19.7	31	43
34	7-8	440-450	178	50	1.0	4.5	31.2	35	37
35	12-13	442-450	136	60	0.4	1.3	8.2	36	90
36	10-11	440-450	178	30	0.8	3.2	31.9	53	38
37	7-8	450-470	268	20	0.7	4.0	42.2	34	5
38	5-6	450-470	268	20	0.9	3.8	40.2	28	4
39	4-5	450-472	280	25	0.7	3.2	36.0	30	22
40	8-9	450-470	268	30	0.9	3.6	38.3	19	5
41	6-7	450-470	268	50	0.8	4.3	45.0	24	5
42	11-12	450-470	268	30	0.7	2.8	30.6	30	8
43	9-10	450-470	268	25	0.7	3.8	40.2	22	4
44	13-14	450-472	283	25	0.7	2.1	25.2	21	17
45	10-11	450-470	268	30	0.8	4.0	42.2	52	22
46	12-13	450-470	268	30	0.7	2.0	22.9	70	20
47	3-4	450-472	283	25	0.6	2.3	27.2	13	19
48	1-2	Rock-470	170	60	0.2	1.1	9.0	17	28
49	2-3	450-470	270	60	0.5	1.3	16.2	61	24
50	14-15	450-472	282	40	0.6	1.8	22.1	21	21
51	15-16	Rock-470	202	60	0.2	1.6	14.3	92	42
52	16-17	Rock-470	128	60	0.1	0.6	4.5	12	8

1/ Opening movement is that indicated by dial gauges at D/S face by grouting pressure tabulated.

2/ Total cement accounted includes all waste, bleeding and grout to fill external lines, as well as cement in joint and embedded system.

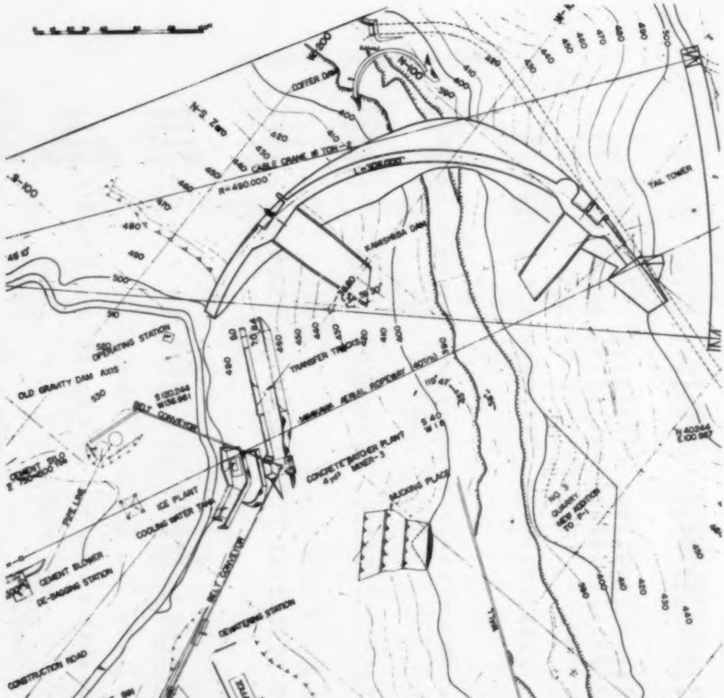

COFFER DAM
CABLE CRANE
TAIL TOWER
OPERATING STATION
OLD GRAVITY DAM AXIS
TRANSFER TRACKS
AERIAL ROPEWAY
CONVEYOR
CEMENT SILO
PIPE LINE
ICE PLANT
COOLING WATER TANK
CONCRETE BATCHER PLANT
MIXER-3
MUCKING PLACE
CEMENT BLOWER
DE-BAGGING STATION
BELT CONVEYOR
DEWATERING STATION
CONSTRUCTION ROAD
SURGE BIN
STOCK PILE
WALL
MOTOR POOL
HOPPER
NEW BRIDGE
CONCRETE LABORATORY
INCLINED RAILWAY FOR AGGREGATE TRANSPORTATION
NO.2 STATION
NO.1 STATION
UNLOADER
WATER TANK
TO QUARRY AREA
RIVER

PLATE NO. I

FOR SECTIONS SEE PLATE NO. 4

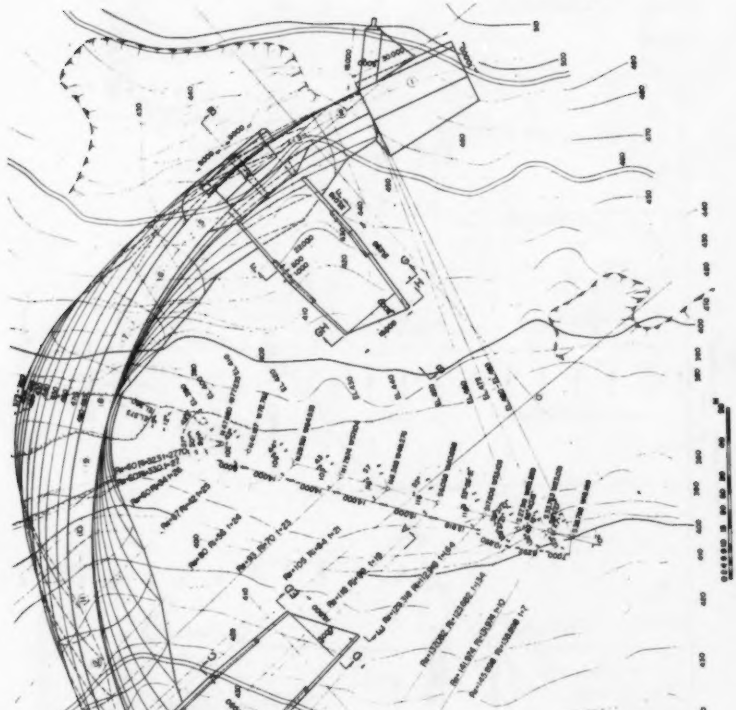


PLATE NO. 2

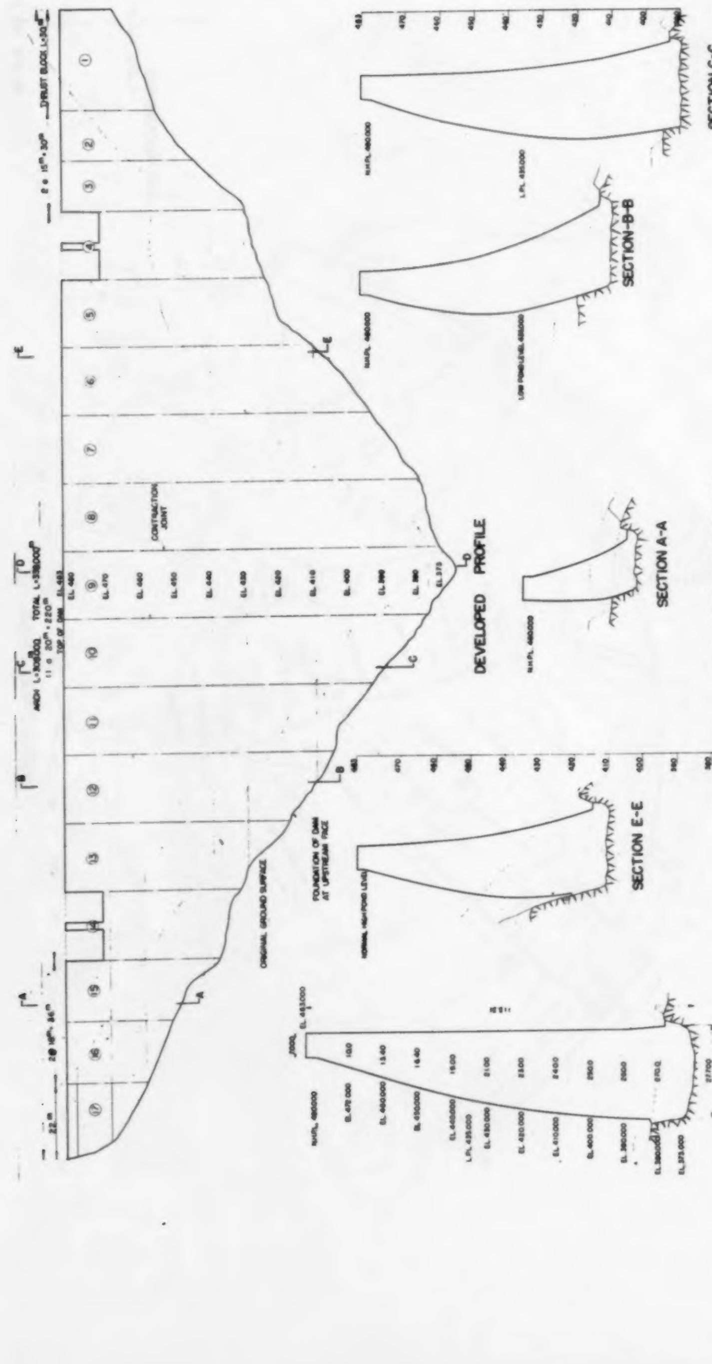
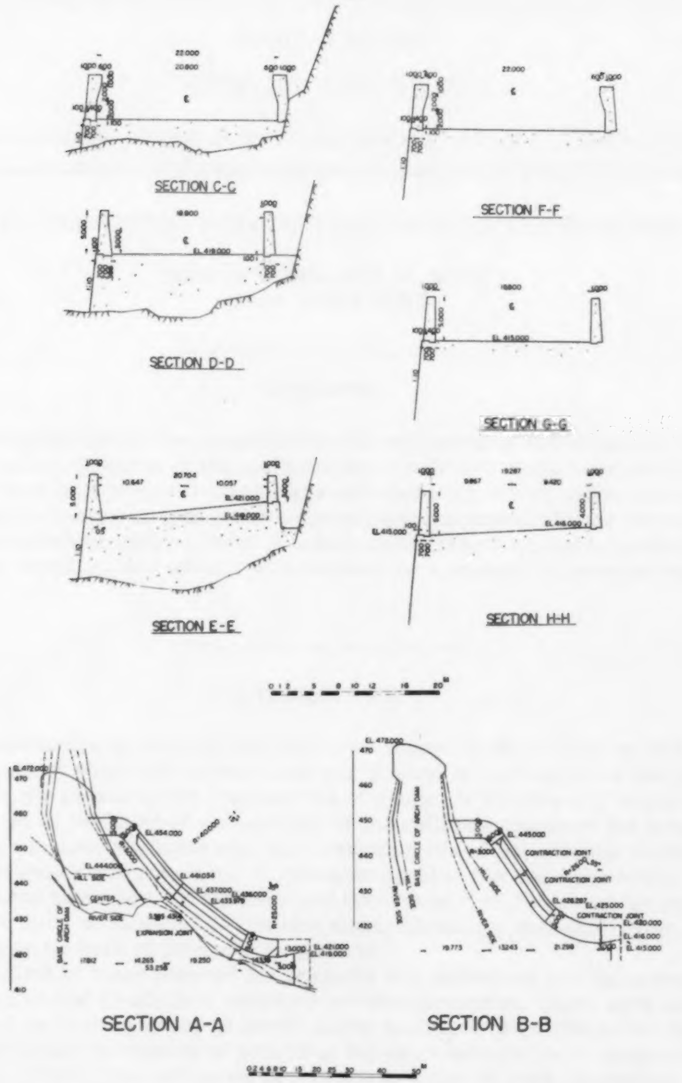

DEVELOPED PROFILE
CONTRACTION JOINT
ORIGINAL GROUND SURFACE
FOUNDATION OF DAM AT UPSTREAM FACE
TOP OF DAM
THRUST BLOCK
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E

PLATE NO. 3



FOR LOCATION OF SECTIONS
SEE PLATE NO. 2

PLATE NO. 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

A SUPPLEMENTAL NOTE ON VALUATION AND DEPRECIATION

Maurice R. Scharff,[1] M. ASCE
(Proc. Paper 1184)

SYNOPSIS

In this paper the author supplements his two previous papers on the subject by calling attention to the applicability of calculations of equal sums of present worths of annual fixed charges and operating expenses for alternative proposals to the solution of other engineering-economic problems, as well as to the valuation and depreciation of public utility property; and by presenting algebraic formulae and numerical examples for a number of selected typical cases.

INTRODUCTION

The author has previously presented two papers to the Society on this subject, in both of which his presentation was limited to exploration of the proposal that the valuation and depreciation of public utility property might be determined on the basis of calculations of the estimated sums of the present worths of amounts available over the remaining life of the existing facilities for amortization and return out of revenues equal to the costs (including amortization and return on the required investments) of rendering the service by the most economical alternative means available, including those available as a result of progress in the arts.

In the first of these papers[2] the proposal was developed and discussed, and formulae and illustrative calculations were presented, based upon comparison of an existing group of public utility facilities, with alternative or substitute facilities capable of providing the same service more economically, giving effect to the estimated cost of construction of such alternative facilities, and to the excess of the operating and maintenance expenses of the

Note: Discussion open until July 1, 1957. Paper 1184 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 1, February, 1957.

1. Cons. Engr., New York, N.Y.
2. Maurice R. Scharff—Valuation and Depreciation of Public Utility Property, Transactions 114, 1949, p. 907.

existing facilities over the estimated operating and maintenance expenses of the more efficient alternative facilities.

In the second of the earlier papers,[3] attention was called to the fact that the effect of income taxes, which are equivalent to additional fixed charges on investments, had been left out of account in the first paper. Formulae and illustrative calculations were presented, giving effect to income taxes as well as to amortization, return and excess operating and maintenance expenses. It was pointed out, also, that property taxes and property insurance costs were based on assessments and insurance values of uncertain relation to valuations on the basis proposed in the paper; but that, if they were considered to be related to such valuations, they could be taken into account by a procedure precisely analogous to that suggested for giving effect to income taxes, requiring only the addition of appropriate terms to the applicable formulae.

Since the presentation of the two papers previously referred to, the author has had occasion to utilize similar calculations in connection with the analysis of a variety of engineering-economic problems, and to observe the consideration, or lack of consideration, given to such calculations in public utility valuation and rate cases, court and commission proceedings and opinions, tax assessment cases, comparison of engineering alternatives and contract negotiations. These experiences have suggested to him that the supplemental observations in this paper might invite criticism and discussion by interested members, with useful results for the profession.

Supplemental Observations

1. All of the discussions, calculations and formulae in the earlier papers were presented from the narrow point of view of their relation to the valuation and depreciation of public utility property. Actually, the procedure of comparing the sums of the present worths of the total annual costs, including amortization, return and operating and maintenance expenses, over appropriate capitalization periods, for two or more alternative capital investments has broad, or even general application to the solution of engineering-economic problems. In addition to the valuation and depreciation of public utility property, other examples of such applications are the selection of the most economical solution among two or more alternatives; the justification of the investment of a capital sum; the appraisal of water power rights by comparison with the cost of steam electric power; etc., etc. The classic example, of course, is the equating of a sum of money to the sum of the present worths of the interest and amortization of a bond, as reflected in the bond values calculated in bond tables.

2. In many cases, where the valuation or justifiable investment for one facility or group of facilities is calculated by equating the sum of the present worths of its amortization, return and operating and maintenance expenses to the corresponding sum of present worths for an alternative facility or group of facilities of known cost, adopted as a basis of comparison, and solving for the unknown valuation or justifiable investment, the result ought properly be described as the maximum or minimum valuation, or the

3. Maurice R. Scharff—Valuation and Depreciation Related to Income Tax, Transactions 117, 1952, p. 341.

maximum or minimum justifiable investment. This is true, in such cases, for the reason that, even with equal sums of present worths of annual costs over appropriate capitalization periods, one alternative may be preferable to another, because of considerations whose precise effect cannot be precisely calculated. Thus, for example, the valuation of, or justifiable investment in an existing obsolete steam-electric generating station with low pressure and temperature and a high heat rate, calculated on the basis of a comparison with a modern station with high pressure and temperature and a low heat rate should properly be described as the maximum valuation of, or maximum justifiable investment in the obsolete plant. For even at equal calculated present worths of total annual costs, the modern efficient plant would be preferable because of greater reliability of service, more ready availability of repair parts, smaller labor requirements, etc. All of these considerations would affect judgment as to the valuation of or justifiable investment in the existing obsolete facility, even though their effects could not be precisely calculated.

3. Both of the earlier papers were limited to the consideration of cases in which not only the equal annual payment annuities for amortization and return, but also the remaining annual capital costs for income tax, property tax and property insurance, and the annual operating and maintenance expenses for each of the alternatives compared would be constant throughout the present worth capitalization period. Actually, in some cases—as, for example, for an existing facility, such as a steam-electric generating station, or a manufactured gas plant, utilized at the date at which present worths are calculated on base load service, but which is expected to operate at a declining instead of a constant capacity factor, and/or which is expected to be transferred at a specific future date to peak load or stand-by service—some of the factors in the required calculations may be different in different years, or periods of years, instead of remaining constant. Accordingly, it may be desirable to give effect to these conditions by somewhat different calculation procedures and formulae.

Formulae and Illustrations

This paper summarizes below the formulae developed in the two previous papers for cases in which the various factors in the calculations are constant for the entire present worth capitalization period; and then supplements this summary by presenting the corresponding formulae for several selected cases in which some of the factors are different in different years or periods of years, instead of remaining constant. For purposes of illustration, the arithmetical effect of applying each of the formulae to a single set of numerical data will be shown.

For convenience, the same symbols will be used as were used in the second of the two earlier papers, to the extent that they are applicable. Additional symbols, as well as the numerical data utilized for purposes of illustration will be introduced below as required.

The first four cases were referred to in the two earlier papers, and relate to single capitalization periods, with all factors in the calculations constant throughout the period.

Case I <u>Calculation based on Comparison of Existing Facility with Alternative Facility, and on Constant Comparative Operating and</u>

Maintenance Expenses (giving effect to Amortization, Return and Operating and Maintenance Expenses, excluding effect of Income Tax, Property Tax and Property Insurance)

This case corresponds with the presentation in the first of the earlier papers, and the calculation procedures used were expressed as an algebraic formula in the second of the earlier papers. To summarize:—

Let V_c = value of or justifiable investment in existing facility

E = cost of construction of alternative economical facility

L_c = remaining life of existing facility in years

L_e = service life of alternative economical facility in years

R = rate of return and capitalization rate

A_c = annuity whose present worth at rate R for period L_c is 1

A_e = annuity whose present worth at rate R for period L_e is 1

O_c = annual operating and maintenance expense of existing facility

O_e = annual operating and maintenance expense of alternative economical facility

P_c = present worth of \$1 per annum at rate R for period L_c (equal to $\frac{1}{A_c}$)

Then, to recall the procedures followed in equating sums of present worths of total annual costs:—

$$P_c\ (A_c\ V_c + O_c) = P_c\ (A_e\ E + O_e)$$

Therefore, as presented in the second of the earlier papers:—

$$V_c = \frac{A_e\ E - (O_c - O_e)}{A_c}$$

or in alternative form:—

$$V_c = P_c\ \left[A_e\ E - (O_c - O_e)\right]$$

For purposes of illustration, it is assumed that comparisons are being made between an existing obsolete plant and an alternative modern efficient plant capable of performing the same service; that E is \$1,000,000; L_e, 35 years; L_c, 20 years; R, 6%; O_c, \$150,000; and O_e \$100,000

Then A_e = .06897

A_c = .08718

P_c = 11.4699

and V_c = \$218,000

Case II Calculation on same basis as Case I, except that effect is given to Income Tax as well as to Amortization, Return and Operating Expense, but neglecting effect of Straight Line Depreciation on Income Tax as if Sinking Fund Depreciation and Bond Interest only were deductible (excluding effect of Property Taxes and Property Insurance)

In addition to the symbols used in the calculation of Case I, the following symbols are used in this case:—

Let B = ratio of funded debt to total capitalization

i = interest rate on bonds

I = ratio of income tax to taxable income

and t = ratio of income tax to capital cost

Then

$$t = \frac{I\,(R - B\,i)}{1 - I}$$

Equating the sums of present worths of total annual costs, as before, and solving for V_c, the result as presented in the second of the earlier papers is:—

$$V_c = \frac{E\,(A_e + t) - (O_c - O_e)}{A_c + t}$$

or, in alternative form:—

$$V_c = P_c\,\frac{E\,(A_e + t) - (O_c - O_e)}{P_c\,t + 1}$$

If, in addition to the numerical data previously used for purposes of illustration, it is assumed that B is 50%; i, 3%; and I, 50%, then:—

$$t = 4.5\%$$

$$\text{and } V_c = \$484{,}000$$

Case III Calculation on same basis as Case II, except that effect is given to effect of Straight Line Depreciation on Income Tax (excluding effect of Property Taxes and Property Insurance)

In addition to the symbols used in the calculation of Case II, the following symbols are used in this case:—

Let t_c = ratio of income tax to V_c

t_e = ratio of income tax to E

Then $$t_c = \frac{I\left(A_c - \frac{1}{L_c} - B\,i\right)}{1 - I}$$

and $$t_e = \frac{I\left(A_e - \frac{1}{L_e} - B\,i\right)}{1 - I}$$

Equating the sums of present worths of total annual costs, as before, and solving for V_c, the result, as presented in the second of the two earlier papers is:—

$$V_c = \frac{E\,(A_e + t_e) - (O_c - O_e)}{A_c + t_c}$$

or, in alternative form:—

$$V_c = P_c\,\frac{E\,(A_e + t_e) - (O_c - O_e)}{P_c\,t_c + 1}$$

Based on the numerical data previously assumed for purposes of illustration:—

$$t_c = 2.218\%$$
$$t_e = 2.540\%$$
and $$V_c = \$406{,}000$$

Case IV <u>Calculation on the same basis as Case III, except that effect is given to Property Taxes and Property Insurance at constant ratios to Capital Costs</u>

In addition to the symbols used in the calculation of Case III, the following symbols are used in this case:—

Let p = ratio of Property Taxes to Property Cost

u = ratio of Property Insurance to Property Cost

Then, equating sums of present worths of total annual costs, as before, and solving for V_c, the result, as referred to but not actually presented in the second of the two earlier papers, is:—

$$V_c = \frac{E\,(A_e + t_e + p + u) - (O_c - O_e)}{A_c + t_c + p + u}$$

or, in alternative form:—

$$V_c = P_c \frac{E\,(A_e + t_e + p + u) - (O_c - O_e)}{P_c\,(t_c + p + u) + 1}$$

If, in addition to the numerical data previously used for purposes of illustration, it is assumed that p is 3-1/2% and u, 1/4%, then:—

$$V_c = \$557{,}000$$

The succeeding four cases have been selected as typical of cases in which certain factors in the calculations are different in different years or in different groups of years, instead of remaining constant throughout the present worth capitalization period. The number of such cases could, of course, be multiplied indefinitely, but it is believed that the four cases selected cover a sufficient number and variety of types of cases to indicate the kinds of procedures and formulae applicable to such cases.

The first of these cases is one in which the remaining life of the existing facility consists of two or more periods, in each of which the operating and maintenance expenses of both the existing facility and the alternative, economical facility are constant for the period, but different from the operating and maintenance expenses in the other periods; and in all of which the other factors in the calculation are constant. An example of this kind would be an existing steam-electric generating station, operating, at the date for which present worths are calculated, on base load, but which because of the expected completion of more efficient capacity, is expected to operate for a period after a definite future date at a lower capacity factor, and then for the remainder of its useful life after a later definite future date on peak load or stand-by service at a very low load factor. The formula for this case is developed in Case V below.

Case V <u>Calculation on the same basis as Case IV for an Initial Future Period, to be followed by one or more other Periods with different Operating and Maintenance Expenses in each Period, and with other Factors in the Calculation remaining Constant</u>

In addition to the symbols used in the calculation of Case IV, the following symbols are used in this case:—

Let $L_n = L_1, L_2$, etc., --- L_m = periods in years such that $L_1 + L_2 +$ --- $L_m = L_c$

$P_n = P_1, P_2$, etc., --- P_m = present worths of \$1 per annum at rate R for periods L_1, L_2 --- L_m; respectively

D_n = D_1, D_2, etc., --- D_m = present worth, at date of calculation, of \$1 at rate R at beginning of periods L_1, L_2, etc., --- L_m, respectively

O_{cn} = O_{c1}, O_{c2}, etc., --- O_{cm} = operating and maintenance expenses of existing facility in periods L_1, L_2, etc., --- L_m, respectively

O_{en} = O_{e1}, O_{e2}, etc., --- O_{em} = operating and maintenance expenses of alternative efficient facility in periods L_1, L_2, etc., --- L_m, respectively

Equating sums of present worths of total annual costs, as before, and solving for V_c, the result is:—

$$V_c = \frac{\sum P_n D_n \left[E\,(A_e + t_e + p + u) - (O_{cn} - O_{en})\right]}{P_c\,(A_c + t_c + p + u)}$$

or, in alternative form:—

$$V_c = \frac{\sum P_n D_n \left[E\,(A_e + t_e + p + u) - (O_{cn} - O_{en})\right]}{P_c\,(t_c + p + u) + 1}$$

If, in addition to the numerical data previously used for purposes of illustration, it is assumed that the remaining life of the existing facility, $L_c = 20$ years, consists of three periods, $L_1 = 5$, $L_2 = 5$ and $L_3 = 10$ years respectively; in which the operating and maintenance expenses of the existing facility are O_{c1} = \$150,000, O_{c2} = \$60,000 and O_{c3} = \$15,000, respectively; and the operating and maintenance expenses of the alternative, efficient facility are O_{e1} = \$100,000, O_{e2} = \$40,000 and O_{e3} = \$10,000, respectively, then:—

$$P_1 = P_2 = 4.2124$$
$$P_3 = 7.3601$$
$$D_1 = 1.0000$$
$$D_2 = .7473$$
$$D_3 = .5584$$
$$\text{and} \quad V_c = \$723{,}000$$

The next case is one in which the operating and maintenance expenses, both for the existing facility and for the alternative, economical facility, decline on a straight line from O_c and O_e in the first year of the period L_c to $\frac{O_c}{L_c}$ and $\frac{O_e}{L_c}$ in the last year of the period L_c. That is, utilizing the same symbols as before, $O_{cn} - O_{en} = \frac{(O_{c1} - O_{e1})\,(L_c - n + 1)}{L_c}$.

The formula for this case is developed below.

Case VI Calculation on same basis as Case IV, except that Operating and Maintenance Expenses, both of the Existing Facility and of the Alternative Economical Facility, decline on a Straight Line from the respective Operating and Maintenance Expenses in the first year of the period L_c to $\frac{1}{L_c}$ times the respective Operating and Maintenance Expenses in the first year, in the last year of the period L_c, with the other Factors in the Calculation remaining Constant

Equating sums of present worths of total annual costs, as before, and solving for V_c, the result is:—

$$V_c = \frac{E\,(A_e + t_e + p + u)}{A_c + t_c + p + u} - \frac{(O_{c1} - O_{e1})\,(A_c\,L_c - 1)}{R\,L_c\,(A_c + t_c + p + u)}$$

or, in alternative form:—

$$V_c = P_c\,\frac{E\,(A_e + t_e + p + u)}{P_c\,(t_c + p + u) + 1} - \frac{(O_{c1} - O_{e1})\,(L_c - P_c)}{R\,L_c\,\left[P_c\,(t_c + p + u) + 1\right]}$$

Using the same numerical data as were utilized before for purposes of illustration:—

$$V_c = \$687{,}000$$

The next case is one in which the ratios of interest deductible for income tax purposes, property taxes and property insurance to capital sums all decline on a straight line from the ratios in the first year of the period L_c to $\frac{1}{L_c}$ times the ratios in the first year, in the last year of the period L_c; and in which all other factors in the calculation remain constant. The formula for this case is developed below.

Case VII Calculation on same basis as Case IV, except that Ratios of Interest deductible for Income Tax Purposes, Property Taxes and Property Insurance to Capital Sums decline on a Straight Line from the Ratios in the first year of the period L_c, to $\frac{1}{L_c}$ times the Ratios in the first year, in the last year of the period L_c, with the other Factors in the Calculation remaining Constant

Equating sums of present worths of total annual costs, as before, solving for V_c, and letting

$$K_a = \frac{(A_c + 1)\,(1 + R)^{L_c} - A_c\,L_c}{R\,(1 + R)^{L_c + 1}}\left(\frac{I\,B\,i}{1 - I} - p - u\right);$$

the result is:—

$$V_c = \frac{E\left[(A_e + t_e + p + u) + \frac{K_a}{L_e}\right] - (O_c - O_e)}{A_c + t_c + p + u + \frac{K_a}{L_c}}$$

or in alternative form, letting

$$K_p = \frac{(P_c + 1)(1 + R)^{L_c} - L_c}{R\,(1 + R)^{L_c} + 1}\left(\frac{I}{1 - I} - p - u\right),$$

$$V_c = \frac{E\left[P_c\,(A_e + t_e + p + u) + \frac{K_p}{L_e}\right] - P_c\,(O_c - O_e)}{P_c\,(t_c + p + u) + 1 + \frac{K_p}{L_c}}$$

Using the same numerical data as were utilized before for purposes of illustration:—

$$V_c = \$556{,}000$$

The final case is one in which the operating and maintenance expenses of both the existing facility and the alternative, efficient facility decline on a straight line as in Case VI, and the ratios of interest deductible for income taxes, property taxes and property insurance to capital sums decline on a straight line as in Case VII, with the other factors in the calculation remaining constant. The formula for this case is developed below.

Case VIII Calculation on same basis as in Case IV, excepting that Operating and Maintenance Expenses, both of the Existing Facility and of the Alternative, Efficient Facility decline on a Straight Line as in Case VI; and the Ratios of Interest deductible for Income Tax Purposes, Property Taxes and Property Insurance to Capital Sums decline on a Straight Line as in Case VII; with the other Factors in the Calculation remaining Constant.

Equating sums of present worths of total annual costs, as before, and solving for V_c, letting K_a and K_p have the same meaning as in Case VII, the result is:—

$$V_c = \frac{E\left[(A_e + t_e + p + u) + \frac{K_a}{L_e}\right] - \frac{(O_c - O_e)(A_c\,L_c - 1)}{R\,A_c\,L_c}}{A_c + t_c + p + u + \frac{K_a}{L_c}}$$

or in alternative form:—

$$V_c = \frac{E\left[P_c\,(A_e + t_e + p + u) + \dfrac{K_p}{L_e}\right] - \dfrac{(O_c - O_e)(L_c - P_c)}{R\,L_c}}{P_c\,(t_c + p + u) + 1 + \dfrac{K_p}{L_c}}$$

Using the same numerical data as were utilized before for illustrative purposes:—

$$V_c = \$702{,}000$$

Similar procedures can be applied to an indefinite number of additional cases, but it is not believed that further elaboration along this line would be useful.

One further comment, however, may be justified. The results of the numerical calculations for the cases selected for illustration may be tabulated as follows:—

Case I	$218,000
Case II	484,000
Case III	406,000
Case IV	557,000
Case V	723,000
Case VI	687,000
Case VII	556,000
Case VIII	702,000

With the possibility of reaching such widely different results on the basis of different assumptions, the author might reasonably be asked to express his own opinion as to the relative usefulness of the various formulae. His reply would be that on the basis of his experience, he would consider the formulae for Cases I, II and III as incomplete and, under most conditions, useless. He considers the formulae for Cases IV and V as the most generally useful, and the formulae for Cases VI, VII and VIII generally unusable, because of the more extremely speculative character of the assumptions required for their use. However, there can be no doubt that they embody considerations of theoretical applicability, and in cases where reasonable ground for establishing such assumptions can be found, their use, or the use of some modification of them, may be desirable.

The author would be especially interested in discussions of this phase of the problem by other members who have considered it.

[illegible]

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1185)

Discussion of
"ENGINEERING ADVANCEMENTS AT MC NARY PROJECT"

by Louis E. Rydell and Glenn H. Von Gunten
(Proc. Paper 639)

LOUIS E. RYDELL,[1] M. ASCE, and GLENN H. VON GUNTEN,[2] A.M. ASCE.—The writers wish to express their appreciation for the valuable contributions to the subject which were made by Messrs. Fisch and Roby and by General Walsh. These discussions have brought out significant features and data which add substantially to the coverage of the subject.

McNary is basically a project to improve navigation on Columbia River, and General Walsh's discussion brings out the increasing significance of navigation development to the regional economy. With regard to the planning of fish passage facilities, the writers are pleased that General Walsh has called attention to the major fishery engineering research program initiated by the Corps of Engineers and now under way in the Columbia Basin. The problem of providing adequate passage facilities, upstream and downstream, for the commercially important runs of anadromous fish spawning in the Columbia River and its tributaries is indicated by the fact that the total cost of fish facilities for the Corps of Engineers' projects constructed or proposed for construction amount to some $130,000,000. It should be noted that specific fish passage facilities provided at McNary have been covered more extensively in a more recent paper (Proceedings Paper No. 895, "Fish Passage Facilities at McNary Dam," by Messrs. Von Gunten, MacLean and Richardson).

Hydro-electric power accounts for more than 90 percent of the economic benefits of McNary Project. General Walsh calls attention to the need for development of additional upstream storage projects concurrent with the development of run-of-river power projects such as McNary. Such storage is a necessity in order to permit balanced and effective development of the water resources of a stream such as the Columbia having wide variations in seasonal flow. This problem is under intensive study at this time in connection with a second review of the Comprehensive "308" Report (House Document 531) on the Columbia River Basin by the Corps of Engineers, which is scheduled for completion in the latter part of 1957.

With regard to the power units installed at McNary, Mr. Fisch points out the distinct advantages of the Kaplan type of turbine under the hydraulic conditions and operating requirements existing at McNary. It is interesting to note that similar turbines of increasingly greater capacity have been or are being proposed for installation at The Dalles, Ice Harbor and John Day Projects. The Kaplan units at the Ice Harbor Project will have a rated capacity of 143,000 h.p. under 89 feet of head and a maximum output of 150,600 h.p. Although final selection has not been made, it is anticipated the units at John

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Day will have ratings of about 10 percent greater than Ice Harbor. Mr. Roby provides a most interesting review of the history of the development of the low-head hydraulic turbine in the United States. In the short period of less than 50 years typical design has progressed from an installation of six 57-inch horizontal units operating in tandem required to drive one 2100 KW generator, to a single 280-inch vertical unit capable of developing 150,00 h.p. or 112,000 KW.

The writers consider it a privilege to have had the opportunity to present, in however brief form, some of the high-lights in engineering advancements achieved in the design and construction of McNary Project.

Discussion of
"PRINCIPLES OF FEDERAL HYDRO-ELECTRIC POWER DEVELOPMENT"

by William Whipple
(Proc. Paper 739)

WILLIAM WHIPPLE.[1]—The excellent discussion of the paper "Principles of Federal Hydro-Electric Power Development" by Mr. Brudenell of the Tennessee Valley Authority brings out clearly a difference in concept as to the basic economic criterion of feasibility. Both concepts require economic comparison of the proposed hydro-electric plant to a possible alternative; but Mr. Brudenell (and the Tennessee Valley Authority) compare it to a hypothetical Federal steam plant, whereas the more complex approach proposed by the author for use generally throughout the United States requires its comparison to a potential private steam plant, with allowance for differences in tax situations. Admittedly, Mr. Brudenell's approach appears to be much simpler, and it can be used in the TVA area, where Federal steam plants are actually built. For the rest of the country, as long as Federal steam plants are not an actual alternative, it does not appear to be realistic to use them as a yardstick for determining the worth of other forms of electric generation. Rather, these alternative forms should be compared with each other, within the context of other actual or potential circumstances.

It is certainly true, as Mr. Brudenell indicates, that the tax comparison is a most difficult one. In fact, his emphasis on Federal income taxes does not adequately recognize the real complexities of the tax structure on power companies, which differs moreover in various parts of the country. However, the difficulties in determining taxes paid in various areas are less than he supposes, by reason of the great amount of specific data on this subject available through the Federal Power Commission. It would be grossly unfair to the private power companies to overlook such tax differences, as used to be done in past Federal practice. The TVA approach of comparison of proposed Federal hydro plants with cost of Federal steam plants as a basis, which Mr. Brudenell advocates, does in fact indirectly prevent this inequity under conditions prevalent in the Tennessee Valley, as far as the hydro plants are concerned, since the Federal steam power used as a basis of comparison is so cheaply produced as to preclude adoption of uneconomic hydro plants. However, it would appear to the author that the tax differential in all of its intricacy would still have to be faced in determining whether the Federal steam plant itself was economically feasible and the economic extent of proposed development.

Mr. Brudenell apparently did not understand the point the author was attempting to make regarding conflict of interest between flood control and power. In the eastern part of the United States, flood control and power production are largely competing interests insofar as storage is concerned. Mr. Brudenell aptly points out an exception in that on the main stem Tennessee the TVA has obtained considerable economy in combining the two, particularly

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as regards effect on Mississippi floods. The generalization of the basic paper is designed to correct the frequent and fallacious impression that storage for power purposes will automatically provide reliable flood control, which impression is generally only correct, even in a limited sense, in certain parts of the United States and under very limited conditions. Reservation of flood control and power storage separately in the same large reservoir is, of course, often feasible throughout the United States; but it is only where, and to the extent that flood flows can be predicted far enough in advance to allow for planned drawdowns that dual purpose storage can be utilized with reliable effect. Such conditions are not generally found in the eastern part of the United States, other than on the St. Lawrence, the main stem Mississippi, and, to a lesser extent, a few other very large rivers. As regards streams of lesser size, the regional differences are quite pronounced.

Mr. Norwood's treatment of the author's paper consists largely of the concise statement of a number of points of disagreement. If the policy points made by Mr. Norwood were accepted as Federal policy, they would have the effect of further stimulating the entry of the Federal Government into the power field by:

a) Neglecting the tax factor in economic comparisons of proposed power developments.
b) Amortizing certain power costs over periods up to 150 years, rather than the shorter periods now used by Federal agencies.
c) Lower interest rates than now used.
d) Reducing allocations of cost to power in multiple purpose projects.

The author does not agree with certain engineering and economic implications of the above. For example, the amortization of Federal power investment over periods longer than now used, up to 150 years, appears to be counter not only to usual expectations as to absolescence of structure, but also to reasonable prudence in the light of potentialities of nuclear power development. In general, Mr. Norwood's views are so divergent from the approach of the basic paper that it is understandable that he should feel that the entire subject might well be reopened.

The author did not, as suggested by Mr. Norwood, overlook early views of the Corps of Engineers as to partnership proposals, including the one cited which was expressed in 1909, and other similar statements even earlier. Such views were not cited because, in the first place, they were considered too remote in time to be pertinent to modern conditions and, secondly, it was no part of the purpose of the paper either to advocate or to evaluate current proposals for changes in Congressional policies as to partnerships or any other subject.

As a Federal official, the author had considered that, in presenting a paper on current aspects of Federal hydro-electric power development, he might encounter critical discussion representing the traditional opposition to such development. It is hoped that the lack of such discussion indicates an improved climate of understanding which will facilitate objective discussion of the unresolved engineering and economic problems in this field.

Discussion of
"FISH PASSAGE FACILITIES AT MC NARY DAM"

by Glenn H. Von Gunten, Hugh A. Smith, Jr. and Berton M. MacLean
(Proc. Paper 895)

GLENN H. VON GUNTEN,[1] A.M. ASCE, HUGH A. SMITH, JR.,[2] J.M. ASCE, and BERTON M. MAC LEAN.[3]—The authors wish to extend their sincere appreciation for Mr. Bell's discussions which add valuable information and suggestions concerning the planning and design of fish passage facilities. Mr. Bell has pointed out some of the salient factors which should be given full consideration by those using the criteria and information presented in the paper concerning the McNary Dam Fish Passage Facilities.

Since writing of the paper, considerable work has been accomplished under the Corps of Engineers' research program under contract by the various fishery agencies. One of the most significant developments in the program is the completion of an extensive experimental fishladder located at Bonneville Dam in which tests are presently being conducted. It is hoped that the experiments in this facility and on other items in the research program, will bring forth factual data leading to improvement in efficiency and economy of upstream migrant passage facilities and replacing some of the judgment factors presently used.

To indicate the nature of the studies and the extensiveness of the research program, an outline of the specific items is given below:

TECHNICAL RESEARCH PROGRAM

1. Criteria for fishladder design
2. Effect of Dams on Rate of Upstream Migration
3. Enumeration of Fish at Dam Sites
4. Criteria for Artificial Spawning Ground
5. Methods of Supplying Auxiliary Attraction Water
6. Endurance and Speed of Fish
7. Efficiency of Submerged Orifices
8. Survival of Downstream Migrant Fingerling at Bonneville & McNary Dams
9. Sonic, Light, Electricity and other means of Guidance
10. Effect of Electricity on Fingerlings
11. Distribution and Time of Migration of Fingerlings in Rivers and Forebays
12. Efficiency of Water Diversion Screen

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2. Hydr. Engr., Walla Walla Dist., Corps of Engrs., U. S. Dept. of the Army, Walla Walla, Wash.
3. Biologist, Walla Walla Dist., Corps of Engrs., U. S. Dept. of the Army, Walla Walla, Wash.

This program is administered and supervised by the Corps of Engineers with the advice and assistance of administrative and technical advisory committees comprised of the interested State and Federal fish and wildlife agencies.

Discussion of
"ARCH DAMS: THEORY, METHODS AND DETAILS OF JOINT GROUTING"

by A. Warren Simonds
(Proc. Paper 991)

JUDSON P. ELSTON,[1] M. ASCE.—I think Mr. Simonds has presented an excellent general coverage whereby in an arch dam it becomes possible to develop arch action prior to any appreciable raising of the reservoir by creating an "arched" monolith. I wonder if in a sense of the word, successful grouting of the joints in an arch dam does not create a "prestressed" arch. Is not the dam made monolithic and does not the equal and even opening of each joint and filling with a dense cement mixture result in a ring in which the blocks are theoretically in tension? Then as the reservoir is filled and load is applied to the ring, would not a slight deflection downstream take place and the end loading be compressive?

The background and history of joint grouting in connection with arch dams is very interesting and informative. Mr. Simonds states on page 991-5 that, "The joint keys developed by the Bureau of Reclamation have now been standarized and are used in the construction of all solid concrete dams." I presume he means only arch dams and not necessarily all gravity dams. I think that within the last five years, considerable study has led some designers to the conclusion that, dependent on the results of studies of topography and shape of the dam site foundation as well as the foundation material itself along with a trial load analysis of the structure, keyways and/or joint grouting may not be necessary or required on gravity type dams. Perhaps Mr. Simonds could enlighten me a little on this subject.

On page 991-7 under the section on "Theory of Grouting Contraction Joints," Mr. Simonds states that high tensile cantilever stresses may develop on the downstream side of the structure, and; "These can be controlled to a large extent by a partial filling of the reservoir to balance the resultant of the grout pressure."

I agree that a partial filling of the reservoir will tend to balance the resultant of pressures but feel that such a procedure is not only difficult to achieve in practice but can become dangerous to the success of the entire grouting operation. Actually, in theory to correctly counteract such pressures, the filling of the reservoir should be taking place at the same time that grouting pressures are being applied to the joints, should it not? This is manifestly almost an impossibility at times during the construction stage for one reason or another. From the practical point of view, might it not be just as satisfactory to cut down on the height of lifts to be grouted? Unless it is possible to have sufficient and exact load acting on the upstream face of the structure to restrain any upstream movement as shown on Figure 3, the joints will be in compression at the downstream face and in tension at the upstream face during grouting. As Mr. Simonds states the adjacent joints

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should be full of water under pressure if necessary. Also the enclosing metal grout stops should be correctly designed and carefully embedded in concrete so as not to rupture and leak under pressure. A theodolite should be set up for measurements on the crown cantilever and dial gauge installations made on the up and downstream faces. By cutting the grouting lift heights from 50 feet down to 30 to 35 feet and with extreme care and precaution (close control) during the grouting operation, I would think it possible to secure the required opening distributed horizontally and vertically without raising the reservoir and without developing undue stresses. I do not take exception to Mr. Simonds' statements! I merely wish to point out that in some cases it has not been possible or necessary to raise the reservoir and that, in fact, raising of the reservoir prior to grouting in itself does not insure that unbalanced stresses or strains will not develop.

Discussion of
"ARCH DAMS:
DESIGN AND OBSERVATION OF ARCH DAMS IN PORTUGAL"

M. Rocha, J. Laginha Serafim, and A. F. da Silveira
(Proc. Paper 997)

FRED A. HOUCK,[1] M. ASCE.—This paper together with all of the other papers presented in the Symposium on Arch Dams of the ASCE, Power Division, in June 1956 are indeed milestones in the evolution of the design and construction of curved masonry dams. The influence of this particular paper will undoubtedly be felt among many of the designers of arch dams of the future throughout the world, especially as regards designing on the basis of model tests. The favorable comparison of results between the observed behavior of the prototype structures and the model tests of the Portuguese dams demonstrates that the Laboratoria Nacional de Engenharia Civil, of Portugal has attained a commendable status in the science and technique of model testing.

The writer is interested in the experience of other engineering organizations besides his own on the matter of analyzing and designing curved masonry dams by means of the "trial load" method of analysis. Having worked with and assisted in the development of the method, the writer can appreciate the author's conclusion that the trial load method in its full application is very laborious and lengthy. However, aside from this criticism, which will be discussed later as a relative point of view, no one has to date demonstrated that the method is inadequate for design. Therefore, the statements made in the last paragraph on page 5 of the paper may appear misleading to designers and engineers not having had much experience in evaluating the results of more than one method of analysis. Also, the author's conclusion that the trial load method, based on a single radial adjustment of the arch and cantilever displacements, can only be used for an estimate of the stresses in a preliminary design is inconclusive to an engineer who understands the relative effects of tangential shear and twist in the overall evaluation of an analysis. It may therefore be well to discuss at this point a number of general tendencies and effects that have been observed from a large number of Bureau of Reclamation and other dams analyzed by condensed and by complete trial load methods. Neglecting those zones of an arch structure where irregularities of base and profile occur and where localized stresses will exist, the following general effects of tangential shear and twist are evident:

1. The maximum stresses in both arch and cantilever elements in most parts of a structure are reduced, except perhaps in the top arch elements where the effects due to thrust sometimes cause a tendency toward tensile stresses in the abutments and at the crown intrados.

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2. In the cantilever elements there is a tendency toward increased compression at the upstream face and a corresponding decrease of stresses at the downstream face, especially in the lower elevations of a dam where the effects of twist are invariably greater.

3. In the central part of a structure, reduction of both radial deflection and stresses is general. The changes of stresses at and near the crown point of arch elements are generally small in comparison with greater reductions at the abutments. At the abutments the tendency is toward an increase in compression at the extrados and a decrease in compression at the intrados while at the crowns of the arches near midheight the opposite effects are usual.

4. In dams where the abutments have been sharply thickened by the addition of fillets, the stresses in the arch abutments will increase considerably due to tangential shear effects. It is impossible to compute these shear effects by means of a radial adjustment of deflections only, but they can be estimated by comparison with the computed effects from other comparable studies.

The Bureau has been able to save much time, effort, and expense in design costs by the judicious application of the above generalities to obtain final or close to final design dimensions. The conjugate points of the arches and cantilever elements representing the structure of a particular design are adjusted into radial deflection agreement by trial load process. The initial and succeeding design dimensions are altered to yield approximate desired stresses in accordance with the above characteristics applied to the results of analyses made to account for only the radial adjustment of deflections in the arches and cantilevers. The final analysis is made to complete the stress study and is usually made with a simultaneous adjustment of radial, tangential, and twist effects on representative elements of the structure.

The cost of making the analyses of stresses sometimes amounts to a large share of the total cost of designing an arch dam. The costs of analyses vary with the complexity of the method used, the number, time, and pay scale of the people employed, and to a lesser degree with the cost of materials, equipment, or other services involved. Contrary to the apparent experience of the Laboratoria Nacional de Engenharia Civil, Portugal, the Bureau of Reclamation has found that design of arch dams by trial load methods yields the most reliable results in less time and cost than by model tests. The question of reliability of results has heretofore been involved in the Bureau's experience because the matter of model testing has not been as conclusive by comparison with computed and observed results as has Portuguese and other European experience along these lines. In verifying the fundamental accuracy of new and existing theories, the use of models took an important part in the solution of many hydraulic and structural problems in the designs of Hoover, Grand Coulee, and other Bureau dams. In contrast to the hydraulic models, which provide direct empirical data, the principal function of the structural models of these dams was to furnish a check on analytical methods of design. Although considerable information which could not be readily obtained by analytical methods was derived and used in design from the arch dam model tests, this was only incidental to their use in determining the adequacy of the trial load method of analysis. From this viewpoint, the method of applying results of structural model tests of dams by the Bureau of Reclamation

differed somewhat from the use of other types of structural models.

Now, however, the authors have contributed valuable evidence to the profession which demonstrates that with careful simulation of all known dimensions, loading conditions, mechanical and physical properties of the dam and foundation, and by the skillful employment of testing techniques, model tests may be used as a reliable basis for designing an arch dam.

It is not the purpose of this discussion to unfavorably compare or advocate the use of one means of analysis over another for arch dam design. The writer believes that whenever model tests are to be made the basis of determining stresses or movements for design, that they should always be checked by proven analytical methods so far as possible.

The writer is also interested in exchanging ideas regarding the time and expense involved in designing by use of the trial load method and by the use of model tests as described in Paper No. 997.

With an experienced team of engineers, the Bureau of Reclamation expends an effort averaging 120 man-days to analyze one loading condition on an ordinary arch dam design with variable thickness elements by adjusting to account for radial movements only. By ordinary arch design is meant that the layout is considered sufficiently symmetrical and that no unusual loading, structural, or abutment conditions exist which would prohibit the structure from being analyzed by symmetrical conditions. If the analysis is carried to completion, including the total effects of tangential shear and twist, an additional 280 man-days, average, is required for this further effort. It costs approximately 40 percent more than the above figures to analyze each additional loading condition for the same layout.

Discussion of
"THE EFFECT OF PORE PRESSURE ON
THE STRESSES IN GRAVITY DAMS"

by O. C. Zienkiewicz
(Proc. Paper 1042)

JAMES PARK.[1]—One of the difficulties in applying elastic theory to concrete bodies is that the material itself is not truly elastic but has pronounced creep characterisitcs. This leads to a variation in the apparent value of Poisson's ratio (ν) which, on immediate loading, may have a value of 0.15 to 0.20, but after a period of weeks or months may reduce to a much smaller value. Because of this phenomenon, to assume a value of zero may be just as accurate in the long run as the more usual experimental figures of 0.15 or 0.20, but there is little experimental evidence to act as a guide in this question.

The author's use of the value zero does in no way affect his main theoretical principles and helps to simplify the numerical computations; but in cases where it is felt desirable to use a value of Poisson's ratio greater than zero, the constant $k = \frac{(1 - 2\nu)}{(1 - \nu)}\eta$ (equation 5) must be retained in this form and the stress expressions (13) and (14) written in the form:

$$\theta = 0 \qquad \sigma_y = \frac{wy}{\sin^2\alpha}\left\{1 - \sin^2\alpha\,(1 + \gamma - \eta) - \eta k \sin^2(\alpha - \beta)\right\}$$

$$\theta = \beta \qquad \sigma_y = wy\left\{\eta k \frac{2\sin^2(\alpha-\beta)}{\sin^3\alpha}\cos\alpha \tan\beta - \frac{\gamma \sin(\alpha-\beta)}{\sin\alpha \cos\beta} + \cot^2\alpha\,(1 - 2\cot\alpha \tan\beta)\right\} \qquad (13a)$$

$$\theta = \alpha \qquad \sigma_y = -\frac{wy}{\sin^2\alpha}\left\{\eta k \sin(\alpha-\beta)\sin\beta \cos\alpha + \cos^2\alpha\right\}$$

and

$$\theta = 0 \qquad \tau_{xy} = 0$$

$$\theta = \beta \qquad \tau_{xy} = wy\left\{\eta k \frac{\sin^2(\alpha-\beta)}{\sin^2\alpha} - \cot^2\alpha\right\}\tan\beta \qquad (14a)$$

$$\theta = \alpha \qquad \tau_{xy} = -wy\left\{\eta k \frac{\sin(\alpha-\beta)\sin\beta}{\sin\alpha} + \cot\alpha\right\}$$

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A similar adjustment must be made to the writer's expressions (15) which appear in Appendix II.

To illustrate the effect of a value of Poisson's ratio greater than zero, the results in Table I have been worked out again using a value of 0.16, the mean result of recently published tests.* The results are shown in the table below.

Table I

$\eta = 1$ $\quad \gamma = 2.420$ $\quad \alpha = 40°$ $\quad \nu = 0.16$

β	$\theta = 0$ $-\frac{\sigma_y}{wy}$		$\theta = \beta$ $-\frac{\sigma_y}{wy}$		$\theta = \alpha$ $-\frac{\sigma_y}{wy}$	
0	(1.000)	0.811	(1.000)	1.000	(1.420)	1.420
5	(0.802)	0.646	(0.877)	0.904	(1.514)	1.497
10	(0.624)	0.491	(0.826)	0.880	(1.586)	1.549
15	(0.463)	0.351	(0.838)	0.910	(1.638)	1.586
20	(0.321)	0.230	(0.904)	0.983	(1.666)	1.598
25	(0.197)	0.132	(1.014)	1.090	(1.667)	1.586
30	(0.097)	0.059	(1.155)	1.207	(1.635)	1.550
35	(0.027)	0.015	(1.305)	1.323	(1.558)	1.497
40	(0.000)	0.000	(1.420)	1.420	(1.420)	1.420

Redrawing the vertical stress diagram of Figure 6 using values from the above table, the stress distribution for $\nu = 0.16$ is as shown in Figure 6a.

The discrepancies involved in the assumption of a linear stress distribution are somewhat different from those shown in Figure 8 if Poisson's ratio is greater than zero. For the value chosen here of 0.16, the discrepancies are as shown in Figure 8a.

Figure 8a shows that the discrepancies incurred in assuming a straight-line distribution of stress are greater when Poisson's ratio is greater than zero, especially at the upstream face when the drainage angle β is small. The apparent discrepancy of 0.189wy when $\beta = 0$ is in fact a theoretical value only, the true stress at that point being given by the second of equations (13a). The discrepancy shown in Figure 8a arises from the horizontal body force which exists between the upstream face and the drainage line, an infinitesimal distance away. This body force creates a discontinuity of vertical stress when ν is greater than zero, so that the graph does not return to zero at that point.

For values of β greater than zero an appreciable error still exists in the straight-line assumption. For a practical case it is unlikely that β would be

* "Poisson's ratio of concrete: a comparison of dynamic and static measurements" by J. C. Simmons, Magazine of Concrete Research, London, No. 20, July 1955.

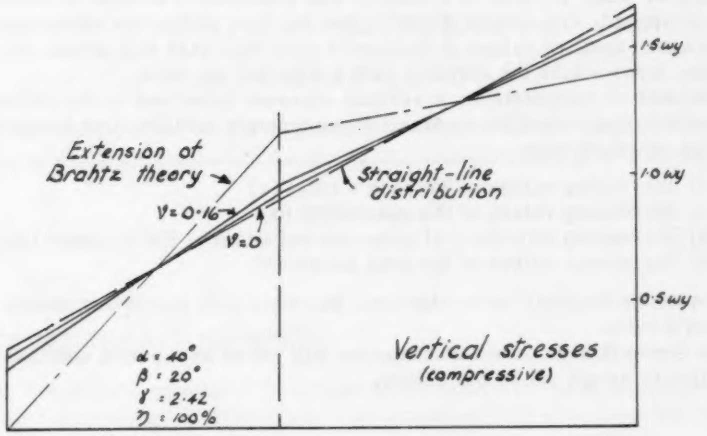
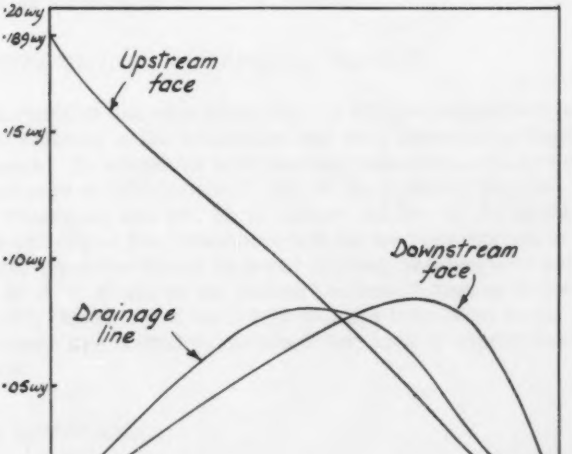


Fig. 6a



Difference between true and linear vertical stresses
(ν = 0·16)

Fig. 8a

less than 5° for an apex angle of 40°. In this case the error in σ_y at the upstream face when $\nu = 0.16$ is 0.156wy. For a section at a depth of 100 feet, this corresponds to a stress of 6.8 lb. per sq. inch so that the discrepancy is not serious. Smaller values of Poisson's ratio decrease this error, for example, for $\nu = 0.10$ the error is only 4.3 lb. per sq. inch.

A number of computations of vertical stresses have lead to the following conclusions; discrepancies incurred by assuming a straight-line stress distribution increase with:

i) increasing values of Poisson's ratio (ν)
ii) increasing values of the apex angle (α)
iii) increasing efficiency of pressure reduction at the drainage line (n)
iv) increasing values of the area factor (η)

Discrepancies in shear force diagrams decrease with increasing values of Poisson's ratio.

It is hoped that the above contribution will serve as a useful addition to the basic theory as set out by the author.

DIVISION ACTIVITIES
POWER DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

February, 1957

NEW COMMITTEE APPOINTMENTS

G. J. Vencill was appointed Chairman of the Power Division Executive Committee and George R. Strandberg was appointed Vice Chairman for the year ending October 1957.

Mr. J. N. Spaulding of Pacific Gas and Electric Company and Asa V. Shannon have been appointed to the Committee on Operation and Maintenance of Hydroelectric Generating Stations. This Committee is now operating under the Chairmanship of Mr. K. O. Strenge of the Washington Water Power Company.

Mr. Adolph Ackerman has accepted the Chairmanship of the Rickey Award Committee replacing Mr. Milton Salzman whose term has expired. Mr. M. P. Aillery has been appointed Chairman of the Publications Committee, and he is now being assisted by Mr. A. V. Dienhart.

COMMITTEE ON NUCLEAR ENERGY FORMED

The purpose of this new committee is "to investigate and disseminate information relating to the economics and civil engineering features of nuclear power plants. To cooperate with similar committees formed by other bodies in the exchange of information." Mr. W. A. Conwell, Duquesne Light Company, is Chairman, and Mr. M. H. Cutler and Mr. F. F. Mautz are members. The first activity of the Committee will be the presentation of a paper "Structural Features of the Waste Disposal System, Shippingport Atomic Power Station" by H. T. Evans at the Second Nuclear Congress in Philadelphia in March 1957. Members of the ASCE who are interested in the development of nuclear power are invited to volunteer for work in connection with this new committee.

FUTURE PROGRAMS

Buffalo Convention June 3-7, 1957. A symposium on penstocks will be presented and 6 papers have been promised by American and French engineers,

Note: No. 1957-4 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 1, February, 1957.

and it is expected that Canadian engineers will present two or more papers on penstocks or allied subjects. Visits will be made to the Huntley Steam Station, Sir Adam Beck Hydroelectric Development with an all-day trip to the St Lawrence development. The charge for the latter including pullman travel from Buffalo both ways at night will be about $30. The full program will appear later in "Civil Engineering".

Annual Convention New York October 1957. An authoritative symposium on underground hydroelectric power plants has been planned, with six papers by eminent engineers, two from the United States and one each from Canada, Sweden, Switzerland and Yugoslavia.

The February 1958 Meeting in Chicago is expected to hear reports of the Committee on Cracking in Masonry Dams, and the Committee on Operation and Maintenance of Hydroelectric Generating Stations. The Committee on Nuclear Energy may also have papers to present at this meeting.

The June 1958 Meeting in Portland, Oregon, will feature a Symposium on Rock Filled Dams being arranged by the Committee on Progress in Power Plant Design.

WELDED SPIRAL CASINGS

The Committee on Progress in Power Plant Design is considering the formation of a task force on welded spiral casings and allied subjects to cooperate with the ASME Hydraulic Prime Movers Committee and with the American Welding Society. ASCE Members who are interested in contributing to this study may contact Mr. G. A. Von Gunten, Walla Walla, Washington for further information.

MEMBERSHIP

The membership of the Power Division has increased substantially since it is possible now to register in two divisions. Members of the Power Division are urged to suggest that other ASCE Members not now registered in two divisions, join the Power Division if they are interested in our field. Papers on power subjects are always welcome either for publication or for convention presentation. Members are also invited to volunteer for committee work with the Power Division since there is always room for active committee participation.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in issue 6 of the Journal of the Hydraulics Division.

VOLUME 82 (1956)

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

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c. Discussion of several papers, grouped by Divisions.

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